CANADIAN THESES ON MICROFICHE

THÈSES CANADIENNES SUR MICROFICHE

National Library of Canada
Collections Development Branch

Canadian Theses on
Microfiche Service

Ottawa, Canada
K1A 0N4

Bibliothèque nationale du Canada
Direction du développement des collections

Service des thèses canadiennes
sur microfiche

NOTICE

The quality of this microfiche is heavily dependent upon the quality of the original thesis submitted for microfilming. Every effort has been made to ensure the highest quality of reproduction possible.

If pages are missing, contact the university which granted the degree.

Some pages may have indistinct print especially if the original pages were typed with a poor typewriter ribbon or if the university sent us an inferior photocopy.

Previously copyrighted materials (journal articles, published tests, etc.) are not filmed.

Reproduction in full or in part of this film is governed by the Canadian Copyright Act, R.S.C. 1970, c. C-30. Please read the authorization forms which accompany this thesis.

AVIS

La qualité de cette microfiche dépend grandement de la qualité de la thèse soumise au microfilmage. Nous avons tout fait pour assurer une qualité supérieure de reproduction.

S'il manque des pages, veuillez communiquer avec l'université qui a conféré le grade.

La qualité d'impression de certaines pages peut laisser à désirer, surtout si les pages originales ont été dactylographiées à l'aide d'un ruban usé ou si l'université nous a fait parvenir une photocopie de qualité inférieure.

Les documents qui font déjà l'objet d'un droit d'auteur (articles de revue, examens publiés, etc.) ne sont pas microfilmés.

La reproduction, même partielle, de ce microfilm est soumise à la Loi canadienne sur le droit d'auteur, SRC 1970, c. C-30. Veuillez prendre connaissance des formules d'autorisation qui accompagnent cette thèse.

THIS DISSERTATION
HAS BEEN MICROFILMED
EXACTLY AS RECEIVED

LA THÈSE A ÉTÉ
MICROFILMÉE TELLE QUE
NOUS L'AVONS REÇUE

Canada

NL 339 (r. 86/09)

THE UNIVERSITY OF ALBERTA

CLAY SOIL-MSW LEACHATE INTERACTIONS AND THEIR IMPLICATIONS IN A LANDFILL LINER DESIGN

by

Ⓒ JOSEPH PATRICK ANTHONY HETTIARATCHI

A THESIS
SUBMITTED TO THE FACULTY OF GRADUATE STUDIES AND RESEARCH
IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE
OF DOCTOR OF PHILOSOPHY

DEPARTMENT OF CIVIL ENGINEERING

EDMONTON, ALBERTA
FALL 1986

Permission has been granted to the National Library of Canada to microfilm this thesis and to lend or sell copies of the film.

The author (copyright owner) has reserved other publication rights, and neither the thesis nor extensive extracts from it may be printed or otherwise reproduced without his/her written permission.

L'autorisation a été accordée à la Bibliothèque nationale du Canada de microfilmer cette thèse et de prêter ou de vendre des exemplaires du film.

L'auteur (titulaire du droit d'auteur) se réserve les autres droits de publication; ni la thèse ni de longs extraits de celle-ci ne doivent être imprimés ou autrement reproduits sans son autorisation écrite.

ISBN 0-315-32592-5

THE UNIVERSITY OF ALBERTA

RELEASE FORM

NAME OF AUTHOR JOSEPH PATRICK ANTHONY HETTIARATCHI

TITLE OF THESIS CLAY SOIL-MSW LEACHATE INTERACTIONS AND THEIR IMPLICATIONS IN A LANDFILL LINER DESIGN

DEGREE FOR WHICH THESIS WAS PRESENTED DOCTOR OF PHILOSOPHY

YEAR THIS DEGREE GRANTED FALL 1986

Permission is hereby granted to THE UNIVERSITY OF ALBERTA LIBRARY to reproduce single copies of this thesis and to lend or sell such copies for private, scholarly or scientific research purposes only.

The author reserves other publication rights, and neither the thesis nor extensive extracts from it may be printed or otherwise reproduced without the author's written permission.

(SIGNED) ....

PERMANENT ADDRESS:

#511, 8515 112 Street..........
Edmonton, Alberta..............
Canada, T6G 1K7................

DATED October 15........1986

THE UNIVERSITY OF ALBERTA

FACULTY OF GRADUATE STUDIES AND RESEARCH

The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies and Research, for acceptance, a thesis entitled Clay Soil-MSW Leachate Interactions and Their Implications in a Landfill Liner Design submitted by J. Patrick A. Hettiaratchi in partial fulfilment of the requirements for the degree of Doctor of Philosophy in Civil(Environmental) Engineering.

....

Supervisor

Frank W. Schwartz

S. Thomson

D.C. Sego

....

External Examiner

Date Oct 10/86

ABSTRACT

A critical evaluation of the current landfill liner design practice revealed that the use of hydraulic conductivity (k) of the soil material as the sole basis for sanitary landfill liner design has serious limitations. In situations where shrinkage cracks are formed, mass transfer of pollutants as leachate from the landfill is controlled by parameters other than the much used hydraulic conductivity value of the clay liner. Cracks may arise from dessication before and/or during landfill operation, synaerisis (resulting from the changes in interparticle physico-chemical forces produced by certain landfill leachate constituents) after the completion of the landfill, or widening of existing dessication cracks resulting from synaerisis shrinkage.

The effect of leachate constituents on soil liner materials was studied to understand the parameters governing the leachate flow from MSW sanitary landfill sites. Factorial design experiments revealed the importance of the multivalent cations (specifically Ca^{+2}) found in leachate, in determining the volume change behavior (shrinking and swelling) of swelling soils. Volume change behavior of bentonites (Na montmorillonite), a clay soil usually recommended as a liner material or as a soil modifier, is greatly affected by Ca^{+2} ions. The concentration of Ca^{+2}, typically found in MSW leachates, is quite capable of causing excessive shrinkage due to synaerisis.

A long term synaeresis consolidation test and a simple index test (sedimentation test) were developed, which are capable of producing parameters to be used in design purposes. The volume shrinkage, caused by a simulated "standard leachate" in the synaerisis consolidation test and the process of sediment formation in the sedimentation test could be adequately explained by the effective stress concept and the electric double layer theories. The reduction in repulsive stresses caused by the double layer diminution could lead to an increase in effective stresses within the soil system resulting in subsequent volume shrinkage, brought about by the loss of water during synaerisis consolidation.

Analytical models based on the effective stress concept/double layer theory and empirical models based on efficient statistical procedures were developed and proposed to explain soil behavior under attack from MSW leachates. A conceptual model of crack formation in liners was proposed. Adaptation of these theories to explain the synaerisis shrinkage, provided a basis to calculate the expected shrinkage of a clay soil liner, and a basis to use sedimentation test in identifying problem soils which are capable of producing excessive volume shrinkage leading to cracking and bulk transport of pollutant.

Methods were proposed to use the developed equations and concepts in a rational clay liner design which accounts for the possibility of crack formation and bulk transport of leachates. Based on the experimental and theoretical evidence a feasible procedure to modify soils, by the addition of Ca ions to the soil, is advocated.

A limited number of low gradient, fixed membrane, direct permeability tests (with provision for volume deflection monitoring) with brine and simulated MSW leachate as permeant, were performed. Results indicated that k increases caused by these waste liquids to be minor. However, synaerisis consolidation contributed to the liquid flow; majority of flow across the sample in the case of brine. Volume shrinkage measurements showed that the possible crack formation due to synaerisis and bulk permeability to be more critical than subtle k changes caused by microstructure changes.

ACKNOWLEDGEMENTS

This research was funded by a grant from the Solid Waste Branch, Pollution Control Division of Alberta Environment. The author also acknowledges the financial support provided by the Civil Engineering Department, the University of Alberta, in the form of assistanships, for the first two years of his PhD program.

The author wishes to express his sincere gratitude to his advisor, Prof. S.E. Hrudey, for his invaluable advice during all stages of this study. Thanks are also due to Prof. D.W. Smith and Prof. D.C. Sego, for their contributions to this investigation.

The cooperation recieved from the environmental and geotechnical laboratory personnel, Mr. R. Billey, Mr. S. Gamble and Mr. G. Cyre, is very much appreciated. The Alberta Environment staff, at the Solid Waste Branch, deserve thanks for the assistance provided in obtaining clay soil samples. The facilities recieved from the Geology Department, University of Alberta and the Research Council of Alberta, in conducting some soil testing are gratefully acknowledged. Furthermore, the author is grateful to Mr. John Fernando, for assisting with the preparation of this manuscript.

Finally, the author wishes to thank his loving wife Srianthi, for providing assistance and moral support throughout the research program.

Table of Contents

Chapter Page

1. INTRODUCTION 1

1.1 STATEMENT OF THE PROBLEM 1

1.2 RESEARCH OBJECTIVES 7

1.3 SCOPE 8

2. LEACHATE MIGRATION CONTROL 9

2.1 CHARACTERISTICS OF MSW LANDFILL LEACHATE 12

2.1.1 Leachate Quantity 12

2.1.2 Leachate Quality 14

2.2 DESIGN OF COMPACTED CLAY SOIL LINERS FOR MSW LANDFILLS 19

2.2.1 Landfill Liner Materials 19

2.2.2 Clay Soil Liners 20

2.2.2.1 General Specifications for Clay Liner Materials 21

2.2.2.2 Recommended Construction Practices 25

2.2.3 Reliability of Design 28

2.2.3.1 Case Study - Clover Bar Landfill Site 29

2.2.4 Limitations in the Design Approach 31

2.3 CLAY LINER PERMEABILITY 32

2.3.1 Permeability and Hydraulic Conductivity ...32

2.3.2 Determination of Hydraulic Conductivity ...34

2.3.2.1 Use of Empirical Formulae 35

2.3.2.2 Laboratory Measurement 37

2.3.2.3 Interpretation of Laboratory Results 41

2.3.2.4 Sample Preparation Methods 43

2.3.2.5 Permeants 44

2.3.2.6 Sample Saturation 44

2.3.2.7 Duration of Experiments 45

2.3.3 Factors Affecting Hydraulic Conductivity of Clay Liners 46

2.3.3.1 Factors Related to the Characteristics of the Soil 46

2.4 USEFULNESS OF LABORATORY DETERMINED k VALUES 52

3. VOLUME CHANGE IN CLAY SOILS AND COMPACTED CLAY LINERS 56

3.1 CLAY-LEACHATE INTERACTIONS 58

3.1.1 Clay Soil and Inorganics 63

3.1.1.1 Important Properties of Clay Soils 65

3.1.1.2 Ion Exchange 67

3.1.1.3 Cation Exchange Capacity of Soils .69

3.1.1.4 Exchange Equations and Ion Selectivity 70

3.1.2 Permeability Changes Caused by Soil-Inorganic Ion Interactions 75

3.1.2.1 Analysis of Case Studies 83

3.1.3 Clay and Organics 84

3.2 VOLUME CHANGE IN CLAY SOILS 91

3.2.1 Mechanisms of Volume Change 92

3.2.1.1 Mechanical Model 92

3.2.1.2 Physico-Chemical Model 93

3.2.2 Effective Stress Concept in Clay Soil-Water Systems 94

3.2.3 The Modified Effective Stress Equation 95

3.3 SOIL-WATER ELECTROLYTE SYSTEM 99

3.3.1 Soil Structure 99

3.3.1.1 Interparticle Forces in Clay-Water Systems 100

3.3.1.2 The Electric Double Layer 103

3.3.1.3 Qualitative and Quantitative Aspects of the Diffuse Double Layer 107

3.3.1.4 Two Particle Interactions 114

3.3.1.5 Determination of R_1 118

3.3.2 Attractive and Short Range Repulsive Forces 122

3.3.2.1 Attractive Forces 122

3.3.2.2 Short-Range Repulsive Forces 126

3.3.3 Interparticle Forces and Clay Swelling ...128

3.3.4 Volume Shrinkage and Crack Formation 131

3.3.4.1 Soil Water 132

3.3.4.2 Mechanisms of Shrinkage and Crack Formation 133

3.3.4.3 Synaerisis Shrinkage 138

3.3.4.4 Effect of Different Fabric Conditions on Shrinkage 139

4. EXPERIMENTAL PROGRAM 141

4.1 INTRODUCTION 141

4.2 PROPERTIES OF THE SOILS STUDIED 142

4.2.1 Clay Minerals 142

4.2.2 Natural Soils 143

4.2.3 Classification Tests 144

4.2.3.1 Clay Minerals 144

4.2.3.2 Natural Soils 144

4.2.4 Exchangeable Cations and Cation Exchange Capacity 147

4.2.5 Soluble Salts 152

4.2.6 Specific Surface Area 154

4.3 LONG TERM SYNAERISIS CONSOLIDATION EXPERIMENTS .158

4.3.1 Synaerisis Consolidation Apparatus 158

4.3.1.1 Loading Unit 159

4.3.1.2 Pressure Cell 162

4.3.2 Test Procedure 163

4.3.2.1 Sample Preparation 164

4.3.2.2 Synaerisis Consolidation Measurements 168

4.3.2.3 Sensitivity of the Synaerisis Consolidation Test Procedure 170

4.4 LONG TERM DIRECT PERMEABILITY TESTS 173

4.4.1 Permeability Testing Apparatus 174

4.4.2 Permeability Testing Procedure 175

4.5 SEDIMENTATION EXPERIMENTS 180

4.5.1 Development Of The Experimental Procedure 180

4.5.1.1 Reproducibility of Sedimentation Test Results 184

4.6 MICROSTRUCTURE STUDIES 186

4.6.1 Scanning Electron Microscope 186

4.6.2 Specimen Preparation And Electron Microscopic Observation 187

5. PRESENTATION, ANALYSIS AND DISCUSSION OF EXPERIMENTAL RESULTS 190

5.1 INTRODUCTION 190

5.2 SYNAERISIS CONSOLIDATION STUDIES 192

5.2.1 Relationship With Leachate Characteristics 193

5.2.2 Effect of Clay Mineral Type and Exchangeable Cations 218

5.2.3 Effect of the External Vertical Stress ...229

5.2.4 Experiments With Natural Soils 236

5.2.5 Effect of Soil Treatment and Time Dependent Changes in Leachate Characteristics 240

5.3 SEDIMENTATION EXPERIMENTS 245

5.3.1 General Introduction 245

5.3.2 Correlation Between Percent Strain and FSDP 251

5.3.3 Relationship Between FSDP and System Variables 262

5.4 HYDRAULIC CONDUCTIVITY AND COMPRESSIBILITY CHARACTERISTICS 274

5.4.1 Calculated Hydraulic Conductivity Results 274

5.4.2 Direct Permeability Test Results 278

5.5 MICROSTRUCTURAL INVESTIGATIONS 279

6. INTERPRETATION AND FIELD APPLICATION OF EXPERIMENTAL RESULTS 282

6.1 GENERAL INTRODUCTION 282

6.2 SYNAERISIS SHRINKAGE AND THE EFFECTIVE STRESS CONCEPT 284

6.3 SEDIMENTATION RESULTS 311

6.3.1 Sedimentation Behavior of Clay Soils 311

6.3.2 Mechanical and Physico-Chemical Force Systems Within a Sedimented Soil Mass 315

6.3.2.1 Effective Stress Concept and Sedimentation 315

6.3.3 Sedimentation and Synaerisis Shrinkage ...318

6.3.4 Interpretation of Sedimentation Results ..322

6.3.4.1 FSDP and System Variables 331

6.4 EFFECT OF LEACHATES ON HYDRAULIC CONDUCTIVITY AND COMPRESSIBILITY 334

6.4.1 Hydraulic Conductivity 334

6.4.2 Soil Compressibility 342

6.5 DIRECT PERMEABILITY, LONG TERM CONSOLIDATION AND SOIL STABILIZATION EXPERIMENTS 344

6.5.1 Direct Permeability Tests with MSW Leachate and NaCl Brine as Permeant 344

6.5.1.1 Brine Permeability 347

6.5.2 Synaerisis Consolidation Experiments at High Pressures and with Brine as the Permeant 353

6.5.2.1 High Pressure Consolidation Experiments 353

6.5.2.2 Synaerisis Consolidation Caused by 30% NaCl Brine 355

6.5.3 Effect of Variable Leachate Composition and Soil Stabilization on Volume Shrinkage 357

6.5.3.1 Effect of Leachate Composition Variations 357

6.5.3.2 Soil Stabilization 360

6.6 IMPORTANCE OF VOLUME SHRINKAGE IN LANDFILL SITUATIONS 364

6.6.1 Crack Formation in Landfill Liners 368

6.6.2 Leachate Movement Across a Liner Containing Macropores 370

SUMMARY AND CONCLUSIONS.................................376

RECOMMENDATIONS 385

BIBLIOGRAPHY 387

APPENDIX A - Notations 407

APPENDIX B - Clay Soils 409

APPENDIX C - Fabric of Clay Soils 420

APPENDIX D - Double Layer Equations 426

APPENDIX E - Time-Settlement Curves 434

APPENDIX F - Void Ratio Vs log Pressure 457

APPENDIX G - Sedimentation Results 497

APPENDIX H - Photomicrographs 506

List of Tables

Table Page

2.1 Leachate Characteristics 16

2.2 Unified Classification System 24

2.3 Potential Errors in Laboratory Permeability Tests 39

2.4 Physical and Chemical Processes in Leachate Migration 42

2.5 Effect of Pore Fluid on Index Properties 50

3.1 Soil - Leachate Interactions 59

3.2 Clay Mineral Characteristics 71

3.3 Hydration Radius of Common Cations 73

3.4 Effect of Increasing Solution Concentration on k 77

3.5 A Comparison of y_c Values 117

3.6 Shrinkage Cracks - Origins and Characteristics 132

4.1 Composition of the Clay minerals 145

4.2 Physical Characteristics of Clay Minerals 146

4.3 Mineralogical Composition of the Native Soils 148

4.4 Physical Properties of the Native Soils 149

4.5 Chemical Properties of the Clay Minerals and Native Soils a) Exchangeable Cations 151

4.6 Chemical Properties of the Clay Minerals and Native Soils b) Soluble Salt Data 153

4.7 Total Specific Surface Area 157

4.8 Results from Replication Studies on Consolidation Apparatus 171

4.9 Results of Replication Studies on Sedimentation 185

5.1 Series #1 Experimental Design 196

Table Page

5.2 Summary of Synaerisis Consolidation Results, Series #1 197

5.3 Series #2 Experimental Design 208

5.4 Summary of Synaerisis Consolidation Results - Series #2 209

5.5 ANOVA Results Obtained for Series #2 Data 212

5.6 Summary of Synaerisis Consolidation Results, Series #3 215

5.7 Relative Influence of Percent Bentonite and Amount Bentonite on Percent Strain 221

5.8 Summary of Consolidation Test Results - Series #5 224

5.9 A Comparison of Percent Strain From Kaolinite/Bentonite and Ca Montmorillonite/Bentonite Mixtures 228

5.10 Percent Synaerisis Shrinkage Strain at Three Different Pressures 231

5.11 Void Ratio Before Leachate Addition 233

5.12 Shrinkage Results for Natural Soils 238

5.13 Synaerisis Shrinkage Results - Series #8 242

5.14 Equilibrium Sediment Volumes With Deionized Water for Some Selected Soils 247

5.15 Calculated C_v and k Values for Series #1 276

6.1 Calculated Half Spacing(d) and Repulsive Stress 291

6.2 Calculated d Spacing and Corresponding Attractive Stresses 304

6.3 Post Lechate C_v Values 339

6.4 Post Leachate k Values 341

6.5 Direct Permeability Results - Standard Leachate 345

6.6 Direct Permeability - Replicate Results 348

Table Page

6.7 Low Gradient permeability Test with Brine - Low Bentonite 349

6.8 Low Gradient permeability Test with Brine - High Bentonite 350

List of Figures

Figure Page

2.1 Schematic Diagram of a Lined Sanitary Landfill 10

2.2 Effect of Compaction on Soil Structure 27

3.1 Forces Acting Within a Clay Soil System 96

3.2 Helmholtz Double Layer 104

3.3 Distribution of Ions Adjacent to a Clay Surface 106

3.4 The Structure of the Double Layer 108

3.5 The Meaning and Variation of ka 112

3.6 Schematic Representation of Repulsion between Clay Particles 115

3.7 Effect of Adding the A Stress 125

3.8 Forces of Interaction 127

3.9 The Clay Micelle 134

4.1 Schematic Diagram of the Experimental Set-Up 159

4.2 Consolidation Apparatus 160

5.1 Time-Settlement Curves for Cell C-1.2 199

5.2 Time-Settlemnt Curves for Cell C-1.5 200

5.3 Effect of Corrosion on Synaerisis Shrinkage 207

5.4 Synaerisis Shrinkage vs Ca Concentration 217

5.5 Percent Strain vs. Percent Bentonite - Bar Diagram 219

5.6 Percent Strain vs Percent Bentonite 225

5.7 Percent Strain vs Amount Bentonite 226

5.8 Synaerisis Shrinkage Vs Void Ratio 234

5.9 Time Settlement Curves for Cell C-8.4 244

5.10 Sediment Volume Vs Ca and Na Concentration 250

Figure Page

5.11 FSDP Vs Ca Concentration 252

5.12 Percent Strain vs FSDP for Series #1 253

5.13 Percent Strain vs FSDP for Series#2 254

5.14 Percent Strain vs FSDP for Series#3 255

5.15 Percent Strain vs FSDP at Different Pressures 257

5.16 Scatter Plot - Percent Strain vs FSDP 259

5.17 Correlation Between Percent Strain vs FSDP 261

5.18 Sediment Volume Vs Mg and K Concentration 263

5.19 Sediment Volume Vs HAc Concentration 264

5.20 FSDP vs Calcium Concentration 270

5.21 FSDP vs Sodium Concentration 271

6.1 Time Settlement Curve for Cell C-4.9 293

6.2 Time Settlement Curve for Cell C-4.12 294

6.3 Void Ratio - Log Pressure for Cell C-4.2 300

6.4 Void Ratio - Log Pressure for Cell C-5.8 301

6.5 Void Ratio-log Pressure Curve for Cell C-4.9 (with A correction) 305

6.6 Crack Formation due to Chemical Action 308

6.7 Sedimentation Curves 312

6.8 Equilibrium Sediment Volume Vs Ca Concentration 321

6.9 Void Ratio - log Pressures for High Pressures 356

6.10 Time Settlement Curve for 3 Series #8 Cells 362

1. INTRODUCTION

1.1 STATEMENT OF THE PROBLEM

Sanitary landfilling is the most popular method adopted for the disposal of municipal solid wastes (MSW). If designed, constructed and operated properly, sanitary landfills allow disposal of solid wastes with minimal environmental disruption.

However, the disposal of huge volumes of solid waste in landfills is not without its environmental impact (Apgar and Langmuir, 1971; Griffin _et al._, 1976). Landfill sites pose a potential threat to surface and groundwater quality. Garland and Mosher (1975) cited several examples of such pollution. The major concern is the generation of leachate, a noxious dark colored liquid containing large amounts of dissolved/suspended organic and inorganic materials, bacteria and trace metals. The chemical strength of leachate generally surpasses that of raw sewage (Lu _et al._, 1985).

The quantity and strength of leachate produced from a given landfill depends upon climatic conditions and landfill management practices. The most common landfill management practices adopted to reduce pollution potential of landfill leachates are: reduction of leachate production, containment of leachate and/or leachate collection for treatment and disposal. Because of numerous recent accounts of groundwater pollution (Griffin _et al._, 1976; Folkes, 1983), installation of a barrier to leachate migration is required whenever a ground or surface water pollution problem is foreseen.

Precipitation in Alberta is low, ranging from 480 mm per annum in the Edmonton area, to less than 300 mm in parts of south eastern Alberta. Of this, about two-thirds falls in the summer when the potential for evaporation is highest. (Davies and Coleman, 1980). Therefore, leachate produced in Alberta landfills will be low in quantity, but high in strength. If allowed to migrate outside landfill boundaries, this high strength leachate could contaminate natural waters. The low quantities and higher strengths make this leachate difficult to collect and treat. The best approach is probably to contain leachate within the landfill and allow microbial decompostion to degrade the organic fraction of the leachate. To succeed with this approach, the landfills must be provided with impermeable liners (natural or engineered) to ensure containment of leachate.

Clay liners are the most common liner type used in sanitary landfills. Clay liners, constructed using native clay soils, are economical and involve well known construction procedures. In addition to leachate containment a liner is capable of attenuating pollutants by sorption, neutralization, precipitation and possibly biodegradation. The variety of functions the clay material can perform favors them over synthetic materials which can also be used as liner materials.

However, to obtain the maximum benefit from clay soil liners, it is necessary to design them properly. Numerous accounts of clay liner failures reported in the literature (Morrison, 1981; Anderson, 1982; and Daniel, 1984) indicate that little attention is given to understanding the behavior and the performance of a clay liner as it exists in contact with leachate.

At present, in Canada, there are no federal or provincial guidelines for the design construction and operation of MSW sanitary landfills requiring compacted fine grained soils as bottom liners. Thus, the design of these MSW sanitary landfills are performed using classical proven techniques applicable to other structures; viz storage reservoirs or lagoons, or using some of the guidelines meant for hazardous waste landfills. For example, the soil material is selected to contain considerable amounts of silt and clay sized particles. This material should be amenable to compaction to a high dry density and a low void ratio, to achieve a very low bulk permeability in the compacted form.

Therefore, all soil materials that can be compacted to a very high bulk density and a very low hydraulic conductivity (usually less than 10^{-9} m/s) are considered liner materials. Initial design of the liner including, selection of the soil material, determination of liner thickness compaction density and water content, are essentially based on a laboratory determination of the hydraulic conductivity (k) of the soil material. If a natural soil is found incapable of showing the desired low hydraulic conductivity, current practice calls for blending with bentonites (containing higher percentages of Na montmorillonite), or providing an additional membrane liner (Clark, 1975).

Various studies have demonstrated the various limitations in this approach. Under normal conditions, laboratory determined hydraulic conductivity values are known to under-estimate the field values by factors of 10 to 10,000 (Dunn, 1983). This discrepency could be attributed to the inability of laboratory determined hydraulic conductivity values to estimate the field bulk permeability.

The main objections to the direct application of laboratory determined hydraulic conductivity values to liner design arise from:

- problems in performing valid permeability tests in the laboratory,
- difficulty in applying laboratory determined values to a non-ideal field situation.

Recently, much research work has been carried out to elucidate the problems associated with hydraulic conductivity measurements (Olson & Daniel, 1981). Dunn (1983) cited a large number of possible errors associated with the hydraulic conductivity measurements for contaminant control. To alleviate these problems, Dunn (1983) proposed a test procedure, together with a new permeability test apparatus (which incorporated a flexible membrane permeameter located within a triaxial cell, with facilities for back pressure and hydraulic gradient application).

However, the hydraulic conductivity measurements obtained by this expensive procedure, too, still tend to underestimate the field values (Dunn, 1983).

The non-ideal behavior of the field situation, where the occurrence of cracks and fissures tend to control the bulk permeability, is the main obstacle to the direct application of laboratory permeability measurements. The smaller samples used in the laboratory do not represent the field macro-fabric at any instance, particularly in landfill situations.

The liner may exhibit dessication cracks formed prior to the commencement of the landfilling of solids. This is common in Alberta, where seasonal climatic conditions necessitate the summer placement of liners prior to winter use. In fact, environmental factors curtail the opportunity of controlling crack development during construction. Therefore, leachate, once formed, would not encounter a structurally sound and a highly impervious barrier as expected. The clay liner with a large number of small cracks or a few large cracks would be susceptible to further attack from infiltrating leachate. The interaction between this not-so-perfect liner and leachate will eventually determine the structural integrity of the liner and its ability to fulfill its requirement as a barrier to leachate.

In situations where dessication cracks are allowed to form before the actual operation of the landfill, bulk flow through opened cracks, rather than the soil parameter k, would control the seepage of leachates across the clay liner. Leachates may either increase or decrease the flow by helping to widen or close these openings. Under certain conditions, the clay soils are known to expand or "swell", and in different circumstances clay soils shrink.

A comprehensive study of the effect of MSW sanitary landfill leachates on the volume change behavior (shrinking/swelling) would shed light upon the expected permeability changes across landfill liners.

Another source of crack formation in a clay liner is the differential volume shrinkage that could occur within the clay liner; this is a direct result of the interaction between the leachate and clay soil. "Synaerisis" action (water loss due to internal physico-chemical mechanisms) is known to cause shrinkage of clay soil and subsequent synaerisis cracks. Leachate with very high electrolyte contents would promote the shrinkage caused by synaerisis action. The significance of this crack formation mechanism, however, depends upon the type of clay soil used in constructing the liner and the leachate generation pattern within the landfill.

Once a clear understanding of the mechanisms involved in the leachate effect on clay liner integrity is obtained, it would be possible to consider them by either modifying the design specifications or by incorporating additional specifications to insure adequate liner integrity.

1.2 RESEARCH OBJECTIVES

The purpose for this study is the development of an understanding of clay liner behavior when in contact with municipal landfill leachates. The findings are directed toward allowing development of design techniques to improve the containment capability of clay soil liners. The main objectives of this study were to:

- identify limitations in the current landfill liner design practice,
- determine the importance of volume change behavior in landfill liner design,
- determine the influence of the characteristics of leachate on soil compressibility (including synaerisis shrinkage) and permeability,
- develop an understanding of the physical basis of synaerisis shrinkage and application of double layer theory to explain this phenomenon,
- develop parameters which can be used directly to identify leachate susceptible soils.

The findings could be used to rationally evaluate the performance of a clay liner in terms of its physical and physico-chemical properties and its response to problems caused by poor construction, inadequate management practices and an aggressive landfill environment.

1.3 SCOPE

To achieve the first two objectives, a comprehensive literature survey was undertaken to review the current landfill liner design practices, mechanisms of volume change in soils (including the available theory to understand this phenomenon), shrinkage and cracking in soils and liquid flow through fissured soils.

The experimental program looked into the influence of leachate on basic soil engineering properties: volume change behavior (shrinkage) and permeability. Sedimentation experiments were used as a means of understanding the effect of leachate on physico-chemical forces within a clay water system, thereby the flocculating/dispersion behavior of clay soil in a leachate solution.

The long term consolidation and sedimentation results were used in the development of soil/leachate parameters which are capable of identifying soil materials that are susceptible to leachate attack. Other soil identification/classification tests, including Atterberg limits, exchangeable cation soluble salt data and microstructural studies were undertaken to prepare a data base and supporting evidence for the final interpretation of the results.

2. LEACHATE MIGRATION CONTROL

Contaminaton of groundwater by landfill leachate is the major environmental risk related to landfilling of municipal solid wastes. Because of the potential risk involved in allowing leachate to migrate beyond the landfill site, best practice calls for reduction of leachate production from landfills and/or minimizing the pollution load leaving the landfill boundary (Figure 2.1).

The most common approaches available for minimizing leachate production and control its subsurface migration are:

- proper site location,
- reduction of surface water inflows,
- use of relatively impermeable soil for intermediate and final cover,
- planting of high transpiring vegetation on completion of landfill,
- use of impermeable membranes overlying the final lift of solid wastes,
- use of subsurface drains and ditches to control the groundwater movement into the sanitary landfill,
- provision of surface and landfill drainage/collection systems,
- use of an impermeable liner to prevent migration of leachate to the underlying soil stratum.

In addition to these, one can rely on the high attenuation capacity of the underlying soil matrix (with or

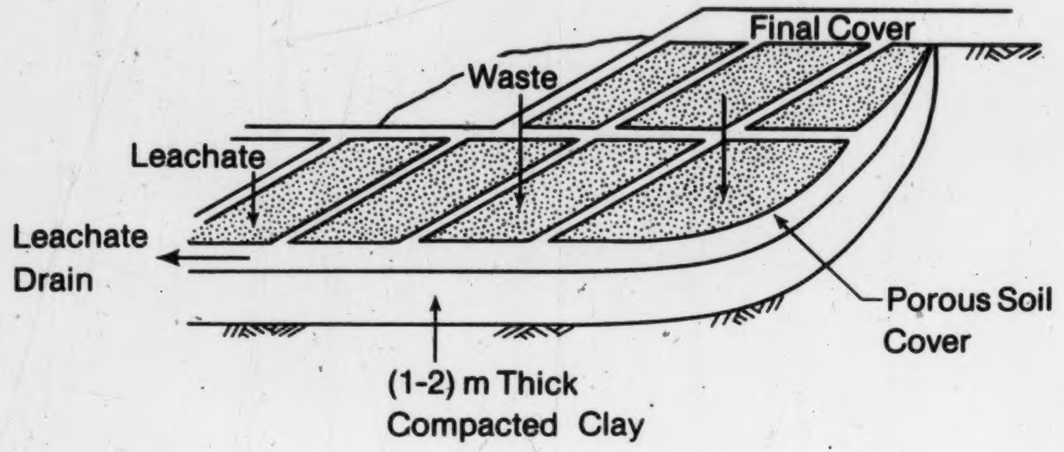
Final Cover
Waste
Leachate
Leachate
Drain
Porous Soil
Cover
(1-2) m Thick
Compacted Clay

Fig 2.1 SCHEMATIC DIAGRAM OF A LINED SANITARY LANDFILL

without a compacted clay soil liner) to reduce the pollution potential of leachate.

The magnitude of the pollution risk posed by the MSW landfill leachate clearly depends on the basic characteristics of the leachate (Fuller and Warrick, 1985). The characteristics, quantity and quality, of leachate produced in landfills vary from one landfill to another and within one site at different stages of landfill operations. Therefore, the selected leachate control mechanisms should be able to cope with the variable nature of this important parameter. The site specific nature of the factors controlling leachate characteristics necessitate an indepth study of each site separately, before deciding on specific mechanisms for leachate control.

The multidisciplinary scope of this study precluded a detailed analysis of the design, construction and operation of landfills in the province of Alberta. However, a good background knowledge of leachate generation and the properties and levels of important constituents (pertaining to this study) was necessary to undertake this study. Therefore, the existing knowledge was reviewed, in order to understand the characteristics of leachate found in Alberta MSW landfills.

2.1 CHARACTERISTICS OF MSW LANDFILL LEACHATE

Leachates are categorized according to the type of waste being landfilled; viz, municipal solid waste (MSW), industrial waste or co-disposal waste. This study is specific to MSW landfills; therefore only the characteristics of MSW leachates will be discussed.

MSW leachate is a relatively dilute aqueous solution of inorganic and organic species, which is generated from water percolating through the waste material. Precipitation, surface run-on and moisture in the waste itself are the general sources of water for the production of leachates. If refuse is buried below the water table, groundwater may also contribute.

The characteristics of the leachate, vary greatly from one landfill to another depending on the solid waste composition, landfill management practices, local weather patterns, hydrogeological conditions at the landfill site and the landfill condition (Johanson and Carlson, 1976).

2.1.1 Leachate Quantity

The leachate generation pattern in a landfill is a function of initial moisture content of the various components, amount of water received from outside sources and the state of biological activity within the fill.

Leachate volume is generally determined by the availability of water, landfill surface conditions and refuse conditions, and the underlying soil matrix (Lu et al., 1985). Factors affecting availability of water include direct precipitation, surface run-on, groundwater intrusion, snowmelt (if collected with solid waste) and refuse decomposition.

Most of the initially generated leachate is squeezed from the solid waste components by the compaction and placement techniques (Fungaroli and Steiner, 1971). The initial production of leachate is low, but once the system has reached its field capacity, production increases to a level approximately equal to the infiltrating water quantity.

The extent of microbial activity and the consequent leachate production depends mainly on the amount and pH of interstitial moisture, temperature, presence of oxygen, composition and particle size of the refuse, organisms present, and the degree of mixing (Lu et al., 1985). Decomposition is much faster under aerobic than anaerobic conditions. Therefore, larger volumes and stronger leachates are common at the beginning of the landfill operation.

In semi-arid Alberta, precipitation rates are low and conditions are not favorable for percolation and generation of large quantities of leachate except for early spring, when the snow melts and the ground thaws. However, if the landfill management practice does not call for daily or even weekly application of cover material, uncovered solid wastes readily absorb direct precipitation or surface runoff and may trap drifting snow. Therefore, significant amounts of water may enter landfill sites which are not properly constructed or maintained (Moell, 1980).

2.1.2 Leachate Quality

Leachates are highly contaminated and can degrade surface water and groundwater. Leachate composition is a function of numerous factors, including those inherent in the landfilling waste and environmental conditions, and those created by landfill designers and site operators.

The chemical and microbiological character of refuse is largely uncontrollable, but the compaction density and permeability of the refuse, depth of the landfill and water application can be regulated. As an example, shredded refuse yields increased landfill field capacity and a high strength leachate. Conversely, baling produces a more dilute leachate. Increasing landfill depth promotes stronger leachate (Fungaroli and Steiner, 1971).

Although, MSW leachates may contain a large number of harmful constituents, in this discussion, only the species that have the potential and are present in sufficient quantity to influence clay soil properties (eg. monovalent and divalent cations, organic acids), will be given full consideration.

Since relatively little water is received from outside sources, little dilution occurs and higher concentrations of organic and inorganic ions are expected in Alberta landfill leachates. Landfills operated as containment structures, using impermeable liners, will further concentrate leachates. Because the leachate remains in contact with the landfilled wastes for a longer time, generated leachates could be extremely high in dissolved salts and organics.

Although interpretation and direct application may not be possible, it is informative to study the composition of leachate samples from different sanitary landfills. Chian (1976), presented a range of values for some of the important parameters and these are compared with values for Alberta landfills as reported by Moell (1980), in Table 2.1. Total dissolved solids in Alberta landfills which ranged from 2000 to 50,000 mg/L, are consistent with other reported results.

Inorganic and organic ions result from:

- dissolution of contaminants from solid waste by contact with leachate solution, and
- acid fermentation of organic matter, which releases large quantities of free volatile fatty acids, while the low pH solution dissolve metals from the refuse.

Quasim and Burchinal (1970) have shown that the concentration of organics is higher early in the landfill life, when the active stages of decomposition occur and organics gradually decrease as the fill stabilizes. On the other hand, minerals may continue leaching for decades.

Chian (1976) reported values of COD ranging from 71,670 mg/L, in an eight month old uncovered landfill containing milled refuse, to 81 mg/L for samples from a landfill

Table 2.1
Leachate Characteristics

Source Parameter (in mg/l except pH and Sp. Conductance)	Range of Values (Chian and DeWalle, 1976)	Alberta Values (Moell, 1980)
COD	40 - 89,520	2013 - 10,246
BOD	81 - 33,360	186.7 - 24,640
TOC	256 - 28,000	3280 - 11,475
pH (pH units)	3.7 - 8.5	5.0 - 13.5
TS	0 - 59,200	not available
TDS	584 - 44,900	2,347 - 51,363
TSS	10 - 700	not available
Sp. Conductance (micromhos/cm.)	2,810 - 16,800	21.23 - 26,230
Alkalinity ($CaCO_3$)	0 - 20,850	656 - 19,603
Hardness ($CaCO_3$)	0 - 22,800	293 - 7,367
NH_4-N	0 - 1,106	6.8 - 1,857
Cl	4.7 - 2,467	708 - 4,520
Ca	60 - 7,200	9 - 3,384
Na	0 - 7,700	840 - 3,470
K	28 - 3,770	726 - 27,776
Sulfate	1 - 1,558	82 - 11,814
Mn	0.09 - 125	0.06 - 45
Fe	0 - 2,820	0.21 - 64
Mg	17 - 15,600	0.1 - 1,069
Zn	0 - 370	0.52 - 87.1

approximately seventeen years old. He analysed leachate samples and found that the majority of the organics in a young landfill were composed of volatile organic acids. The free volatile acids are formed during first stage anaerobic digestion; complex organics are converted to organic acids which are relatively water soluble. Results obtained by Johansen and Carlson (1976) show that, for high strength leachates, the organic acids make up the largest fraction of the total organics. Acetic, propionic and butyric are the dominant acids, contributing almost 90% of the total organic carbon content (Baedecker and Back, 1980). During the acid phase, pH values are usually below 7.0, and ranges between 4.5 and 6.5 (Genetelli and Cirello, 1976). The flow rate of the leachate, through the refuse, controls the character of the leachate pH (Fungaroli and Steiner, 1971). The low pH, common in leachates, causes the dissolvation of trace metals and other ions (Kjeldsen and Christensen, 1984).

The free volatile acid fraction decreases considerably with increasing age of the fill. Organic matter decreases through anaerobic methane fermentation (second stage of anaerobic decomposition) and wash out (Baedecker and Back, 1980). The inorganics decrease more slowly, because washout by the permeating leachate is the major removal mechanism. Chian (1976) measured the concentration of inorganics as fixed solids and found the values to range from a high value of 22,895 mg/L to a low value of 812 mg/L. The organic and inorganic contaminants in leachate are known to go through maxima, and drop to low levels after several years.

Both sulfates and chlorides are found in abundance in the early stages of the landfill. However, because of the reduction of sulfates to sulfides under anerobic conditions, the sulfate concentration decreases with the aging of the landfill.

2.2 DESIGN OF COMPACTED CLAY SOIL LINERS FOR MSW LANDFILLS

As noted earlier, one of the most important modes of minimizing pollution potential of a sanitary landfill is the mitigation of subsurface movement of leachates. In practice, this objective is achieved by providing a highly impervious compacted clay liner below the sanitary landfill.

2.2.1 Landfill Liner Materials

Lining landfills with compacted clay soils have now been demonstrated to be an effective means of preventing leachate pollution of water sources by impeding the flow of the pollutant and the pollutant carrier (Haxo, 1981) and by filtering and/or adsorbing suspended and dissolved pollutants (Pohland and Engelbrecht, 1976). Geswein (1975), in an assessment of liners for land disposal sites, stated that an ideal liner should exhibit the following characteristics:

- low permeability throughout the landfill "active" period,
- capability of sorbing a maximum amount of inorganic and organic constituents from leachate without losing low permeability in the process.

Most liner materials function by both mechanisms but to different degrees (Griffin _et al._, 1976). Their action depends upon:

- the type of liner material, and
- the type/characteristics of waste material being

landfilled.

Synthetic membrane liners can provide a highly impermeable barrier to leachate if properly placed. However, because of the lack of long term exposure experiences, their effectiveness in preventing leachate contamination of water sources in the long run cannot be guaranteed (Matrecon, Inc. 1980). In addition, their comparatively high cost and need for sophisticated placement procedures, have precluded their use in most situations.

Clay liners, using native clay soils, are generally more economical, and involve well known construction procedures. Consequently, clay liners are generally the alternative chosen for sanitary landfill situations. Furthermore, the ability of clay sized particles to remove pollutants by sorption, neutralization and precipitation provide clay liners with several means for preventing the escape of pollutants (Sanks et al., 1975). Given the wide applicability of clay liners to sanitary landfills, only clay liners will be considered in this research project.

2.2.2 Clay Soil Liners

The clay soil liners may be used as a gross filter for certain contaminants and/or as a barrier that contains the waste for some period of time (until the pollution potential of generated leachates have diminished to a lower level).

2.2.2.1 General Specifications for Clay Liner Materials

Unlike for hazardous waste landfills and other types of waste material holding facilities, no specific guidelines are followed by Federal or Provincial environmental agencies in designing clay liners for MSW sanitary landfills.

Moell (1980), investigating Alberta landfills, proposed several site selection criteria for environmental protection and operational efficiency. He proposed to gain maximum use of naturally occurring clay soils, lacustrine clay and glacial till, but did not see the necessity for engineered clay liners. Although these soils are capable of showing very low hydraulic conductivities (usually less than 10^{-8} m/s) in the laboratory permeability tests, field values could be much higher, because of the naturally occurring abnormalities, cracks, fissures, slickensides and highly permeable silt and sand lenses. However, if natural, undisturbed, unfissured clay deposits are found and proper hydrogeological investigations could be performed in order to locate subsurface abnormalities, these clay deposits are preferred over compacted clay liners (Moell, 1980). Nevertheless, the latter are often required, especially if existing excavations are to be used as landfill sites.

Leskiw (1981) suggested to Alberta Environment the use of compacted clay liners, as a means of preventing leachate movement to the groundwater table. He also noted the lack of knowledge in the area of leachate containment technology, which invariably precludes the application of clay liners with confidence in a sanitary landfill design.

Quigley et al., (1984) noted several requirements for the design and construction of compacted clay liners. They proposed a liner thickness of 1.0 to 1.5 m and a hydraulic conductivity, k, with respect to water, of between 10^{-9} and 10^{-10} m/s.

The general trend seems to be to use clay soil material in a highly compacted form to achieve a very low hydraulic conductivity (Fuller and Warrick, 1985). The specifications concerning soil properties used by state regulatory agencies in the U.S. specify tight clays with no sand or gravel seams and a hydraulic conductivity of less than 10^{-9} m/s. (Fung, 1980). Clay soils, with a natural clay content of 25 to 30%, could be mechanically compacted to the desired density to establish hydraulic conductivity below 1×10^{-9} (Fuller and Warrick, 1985).

Regulations under the Resource Conservation and Recovery Act of 1976, drafted by the United States Environmental Protection Agency (US EPA), contained standards regarding the use of lining materials in hazardous waste impoundments (Fung, 1980). Although meant to be satisfied by soil material used in surface impoundments, the following specifications should also apply to soil materials being used in sanitary landfill situations.

They recommended that the soil material should:

- be classified under the unified soil classification system as CL, CH, SC or OH (see Table

2.2 for details),
- allow more than 30% passage through a #200 sieve,
- have a liquid limit equal to or greater than 30,
- have a plasticity index equal to or less than 15,
- should exhibit a hydraulic conductivity not exceeding 10^{-9} m/s, and
- have a pH 7.0 or higher.

Furthermore, compatibility of soil with the waste (in this case the leachate) was considered to be essential, because certain chemicals in the leachate are known to be capable of altering physical and chemical characteristics of the liner. These may allow undesirable permeability changes to occur.

In situations where sufficient clay soil material is not available locally, bentonite (Na montmorillonite) mixed with silty sand at a ratio of 1:20 has been used to produce hydraulic conductivities less than 10^{-9} m/s (Weiss, 1974).

The test techniques necessary to identify clay soil materials for compaction to a low permeable liner are rarely specified in detail by environmental authorities. As such, the design consultants are forced to perform standard tests, for example, standard permeability tests (Quigley, et al., 1984); for the determination of hydraulic conductivity. These tests may not be suitable for the specific requirements of the situation. The soil-waste compatibility testing requires the difficult task of performing laboratory measurements, which frequently require such extensive

TABLE 2.2

UNIFIED CLASSIFICATION SYSTEM AND SOIL CHARACTERISTICS
PERTAINING TO SANITARY LANDFILLS (Modified from, Sittig, 1979)

MAJOR SOIL DIVISIONS	SYMBOL	SOIL TYPE	PERMEABILITY Coeffecient (k) cm/sec	Compaction Characteristics	POTENTIAL FROST ACTION
GRAVEL & GRAVELLY SOILS	GC	Clayey gravels,	10^{-6} to 10^{-8}	Fair, rubber-tired, sheepsfoot roller	Slight to medium
SAND & SANDY SOILS	SM	Silty Sands	10^{-3} to 10^{-6}	Good, with close control, rubber-tired, sheepsfoot roller	Slight to high
	SC	Clayey Sands	10^{-6} to 10^{-8}	Fair, rubber-tired, sheepsfoot roller	Slight to high
Silts & Clays (Liquid Limit less than 50)	ML	Inorganic silts, silty or clayey fine sands, clayey silts	10^{-3} TO 10^{-6}	Good to poor close control essential	Medium to very high
	CL	Inorganic CLAYS of low to medium plasticity, sandy clays, silty clays, lean clays	10^{-6} TO 10^{-8}	Fair to good, sheepsfoot roller, rubber-tired	Medium to high
	OL	Organic Silts and organic silt-clays of low plasticity	10^{-4} TO 10^{-6}	Fair to poor, sheepsfoot roller	Medium to high
Silts and Clays (Liquid Limit greater than 50)	MH	Inorganic silts, micaceous or diatomaceous fine sand or silty soils, elastic silts	10^{-4} TO 10^{-6}	Poor to very poor, sheeps -foot roller	Medium to very high
	CH	Inorganic clays of high plasticity, fat clays	10^{-6} TO 10^{-8}	Fair to poor, sheepsfoot roller	Medium
	OH	Organic clays of medium to high plasticity, organic silts	10^{-6} TO 10^{-8}	Poor to very poor, sheeps -foot roller	Medium

chemical control that they appear more like research projects than practical tests for design. The expensive, time consuming nature of these tests prevent them being used as regular design tools by consulting engineers.

2.2.2.2 Recommended Construction Practices

The permeability across the clay liner depends upon the advocated compaction condition (Wang and Huang, 1984). The required compaction (expressed in terms of the degree of compaction) in order to achieve the necessary low permeability is determined from its maximum dry density. To achieve the required degree of compaction, the molding water content must be near its optimum value (Lambe and Whitman, 1969). Therefore, the maximum dry density and optimum water content are considered essential data for the design of compacted clay liners (Wang and Huang, 1984).

In practice, a standard Proctor compaction test on the selected soil material needs to be performed to obtain this information. The liner is constructed by compaction, using a "wet of optimum" water content, to the identified bulk density. In soil mechanics, the optimum water content is defined as the water content at which a specified amount of compaction produces a maximum soil density (Perloff and Baron, 1976).

The water content principally affects the resistance to relative movement of soil. The resistance to particle movement is provided by the friction between the particles and the attractive and repulsive forces of the adsorbed water layer.

When a relatively small amount of water is present in the soil, it is firmly held by the electrical forces at the particle surfaces. At this stage, the diffuse double layers are not fully developed. Long range repulsive forces are smaller than the interparticle attractive forces. This net attraction does not allow the particles to move over one another easily. Thus, at low water content, less compaction or low dry density with higher percentage of air voids is obtained. An increase in water content tends to lubricate the soil and make the soil more workable, resulting in a higher dry density for the same compactive effort. The increase in dry density, with reduction in air voids, continues until the "optimum water content" is reached.

Beyond the optimum water content, air voids approach approximately a constant value. Further increase in water content does not cause any appreciable change in voids, even though a more orderly arrangement of particles (Lambe, 1958a) may exist at higher water content.

Therefore, compaction at "wet of optimum" water content tends to produce a more orderly, parallel arrangement of particles (Figure 2.2), which is known to give rise to smaller hydraulic conductivity values (Anderson and Glysson, 1982). The soils compacted "dry of optimum" tend to have a more random orientation (flocculated structure); they also have larger pore sizes than do the samples with more parallel orientation of particles (dispersed structure),

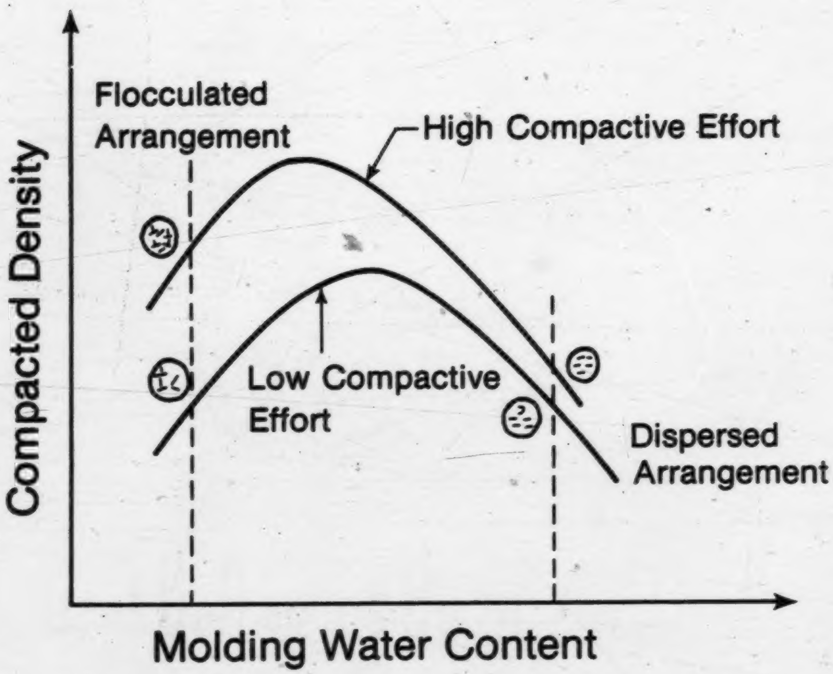
Flocculated
Arrangement
High Compactive Effort
Low Compactive
Effort
Dispersed
Arrangement
Compacted Density
Molding Water Content

Fig 2.2 EFFECT OF MOLDING WATER CONTENT AND COMAPCTION ON STRUCTURE OF REMOULDED SOIL (modified from Lambe, 1958a)

obtained from wet side compaction (Lambe, 1958a). For any given void ratio, the larger the size of individual voids, the greater is the hydraulic conductivity, k.

The development of a dispersed structure in a soil is enhanced by selecting kneading compaction methods in the field. Increased shear forces, created by sheep's-foot rollers, tend to break the bonds between soil particles and produce more orderly orientations. Pneumatic-tired compactors can also be considered because their action is believed to be a combination of pressure and kneading. However, the kneading action of sheep's-foot rollers is known to result in a better bond between compacted layers, compared to pneumatic-tired rollers, and the former are widely recommended for compacting clay soils for landfill liners (Anderson and Glysson, 1982).

2.2.3 Reliability of Design

Detailed case studies of leachate migration from sanitary landfills are not found in literature. However, numerous accounts of leachate migration away from landfill sites have been reported, (Anderson and Dornbush, 1967; Apgar and Langmuir, 1971; Garland and Mosher, 1975), including one in the Clover Bar landfill site in Edmonton (Moell, 1977).

Most of the detailed case studies and scientific research is directed towards understanding the clay liner behavior in hazardous waste landfills, water and brine retention ponds (Daniel, 1984; Haug, 1985; Ridley, 1985). Ridley (1985) studied the long term exposure of clay soils to high strength brine. In performing permeability tests, he observed a "breakthrough phenomenon", where large increases of permeability occurred, not predicted by Darcy's law of water movement through fine grained soils.

Daniel (1984) reported several case studies, where rates of leakage from fresh water storage ponds lined with clay significantly exceeded the rates that would have been predicted on the basis of laboratory permeability tests. The actual hydraulic conductivities of the clay liners were generally found to be 10 to 1000 times larger than values obtained from laboratory tests, on either undisturbed or recompacted samples of the clay liner.

In light of this discussion, higher than predicted leachate leakage rates across compacted clay soils below a MSW sanitary landfill could conceivably occur. Evidence of such action was found in Edmonton and reported by Moell (1977).

2.2.3.1 Case Study - Clover Bar Landfill Site

The precarious hydrogeologic setting of this landfill with respect to the North Saskatchewan River, necessitates the installation of compacted clay dikes (Moell, 1980) between the river and the landfill site. The landfill has been designed to operate as a containment structure, with provision for diversion of groundwater around the landfill site.

Although this containment structure is designed to isolate and contain the leachate within the landfill boundaries, investigators found high concentrations of contaminant of MSW and industrial waste origin 30-60 m outside the landfill boundary (Moell, 1977). During a subsequent examination, Moell (1977) observed breaches in the clay liner, allowing release of leachate to a groundwater intercepter drain in one area, and migration of leachate to a distance of about 60 m from the site in another area.

A constituent analysis indicated that the leachate is entering the groundwater drainage system in small quantities and is invariably diluted to levels below toxic limits. However, Moell (1977) found high concentration "leachate slugs" entering the drain on an intermittent basis, indicating possible failure of the clay liner in one or several places.

Leachate was found to escape from the landfill by several methods. They were:

- by seeping across the clay liner into the groundwater intercepter drain,
- seepage across the clay lined dike situated between the landfill and the North Saskatchewan River,
- seepage through the clay shale bedrock beneath the site via a semi-pervious sandstone layer found in some parts of the site.

2.2.4 Limitations in the Design Approach

The foregoing discussion indicates that any attempt to directly apply the laboratory-determined, hydraulic conductivity values, in predicting field permeabilities, needs to be viewed with caution. In the majority of cases reported in literature, the laboratory determined soil parameter hydraulic conductivity, underestimated the compacted clay liner leakage rate (Dunn, 1983).

However, laboratory-measured hydraulic conductivities are generally used as the basic criterion in designing liners. Specifically, the laboratory permeability tests are used to predict the field hydraulic conductivity of the clay liner. Darcy behavior is assumed and the Darcy formula is applied to calculate the quantity of leakage across the compacted clay. Thereafter, depending on the hydrogeological considerations of the selected site, the final compacted clay liner thickness is decided.

This approach ignores the following limitations:

- problems involved in obtaining valid hydraulic conductivity values in the laboratory,
- inaccuracies which arise from assuming ideal Darcy behavior for a liner which may contain cracks and which may be affected by contact with leachates.

2.3 CLAY LINER PERMEABILITY

2.3.1 Permeability and Hydraulic Conductivity

The effectiveness of clay liners, in preventing leachates migrating outside a landfill site, primarily depends on the permeability of the liner.

Darcy (1856), empirically established the following relationship from some flow tests through coarse grained soils:

$$q = k\ i\ A \qquad 2.1$$

Where, q is flow rate (L^3/T); i is the hydraulic gradient (L/L); A is the total x-sectional area of flow (L^2) and k is the hydraulic conductivity (L/T). This parameter is often referred to as "coefficient of permeability" or simply "permeability", by civil engineers.

For clarity, in this report, k will be called the hydraulic conductivity. Permeability will refer to the general ability of the soil material to transmit water or other fluids.

The hydraulic conductivity or the ratio of the flux to the hydraulic gradient, is not an exclusive property of the soil alone (Hillel, 1971), since it depends upon the attributes of the soil and of the permeating liquid together. The soil characteristic which affects k is the pore geometry of the soil. This geometry is determined by the: total porosity, the pore size distribution and tortuosity of the soil.

If only the characteristics of the porous medium are considered, the constant of proportionality becomes the intrinsic permeability K, which is related to hydraulic conductivity k by the Kozeny - Carmen relationship (Gorden and Forrest, 1981):

$$k = K\ (\gamma/\mu) \qquad 2.2$$

Where, γ and μ are the specific weight and the viscosity of the fluid respectively. In an ordinary liquid, the density is nearly constant, but the viscosity changes with varying solution composition and temperature.

The Darcy's law introduced above, is not valid for all conditions of liquid flow in porous media. Darcy's law is applied to flow relative to the particle, and seems to fail when the particles themselves are moving relative to a fixed frame of reference (Bear, 1972). Darcy's law applies only as long as fluid flow is laminar, as in the case with flow in silts and finer materials (Klute, 1965). Bear (1979) reviewed existing experimental data and found that Darcy's law is valid for a Reynold's number (R_e - a dimensionless number that measures the ratio of inertial to viscous forces during flow), not exceeding some value between 1 and 10. In this range, the viscous forces are predominant.

As the velocity of the flow increases, a region of gradual transition is observed, from viscous, force-controlled, laminar flow to inertial-force governed laminar flow. Often, R_e = 100 is mentioned as the upper limit of this region in which Darcy's law is no more valid

(Bear, 1979). Existence of a lower limit to the Darcy's law, with some threshold gradient necessary to initiate flow has also been reported (Lutz and Kemper, 1959). This is true especially for very small pores (Bear, 1979).

Deviations from Darcy's law (when hydraulic potential gradient becomes less effective in inducing flow), occur at high flow velocities/large pores and very low gradient/small pores situations (Miller and Low, 1963; Lutz and Kemper, 1959). However, Darcy's law can be employed in the vast majority of cases concerning the flow of water in soil (Olson, 1965).

2.3.2 Determination of Hydraulic Conductivity

In the past, many engineers have assumed that fine grained soils are effectively impervious and have foregone attempts to determine hydraulic conductivity (Olson and Daniel 1981). However, when confronted with pollutant movement over periods of hundreds of years, the anticipated low permeability of fine grained soils may not be adequate and hydraulic conductivity should be accurately determined.

The expected conductivity of a clay liner in the field is traditionally obtained from laboratory, clay-soil permeability studies, by the use of empirical relationships or by Terzaghi's method of consolidation calculations.

2.3.2.1 Use of Empirical Formulae

Green et al., (1979), reviewed the empirical formulae available to estimate k in soils. The available equations are based on Poissseulle's model, in which the average flow velocity through a capillary tube is taken as proportional to the diameter of the tube.

Purely theoretical formulae, to calculate the hydraulic conductivity (k) of a soil, have been obtained from theoretical consideration of the Darcy's law. An example is the Kozeny-Carman equation (Bear, 1972), for intrinsic permeability (K) determination:

$$K = C_0 \frac{n^3}{(1-n)^2 \, M_s{}^2} \tag{2.3}$$

Where, M_s is the specific area of the porous matrix (per unit volume of solid), C_0 is a coefficient having a value of 0.2 and n being the void ratio.

Some of the available formulae are empirical. Freeze and Cherry (1979), proposed the following version of Darcy's law, which was derived using results obtained from laboratory experiments performed with an idealized porous medium (chemically inert glass beads or clean homogeneous quartz sand):

$$v = q/A = (Cd_e{}^2)\rho gi/\mu \tag{2.4}$$

Where,

C = the propoprtionality constant or shape factor dependant upon the grain shape, orientation, soil packing and size distribution,

d_e = effective grain size, d_{10},

v = specific discharge, L/T,

g = acceleration due to gravity, L/T^2,

ρ = mass density of the permeant, M/L^3.

Although these equations are found to be adequate in predicting k in cohesionless soils, their applicability to cohesive clay soils is questionable. In coarse grained soils, it is commonly assumed that C and d are characteristic of each soil and once determined, remain constant. Thus variations in K (in Eq. 2.2) are often assumed to depend on the ratio γ/μ. This is a good assumption for sands and silts that are generally inert to most permeating fluids, but fails in the case of fine grained soils containing large amounts of clay particles.

The colloidal material, when wetted, develops electrochemical forces, which, in turn, control the interaction between soil particles and therefore the pore size distribution of the soil mass. Green et al., (1979) suggested that bound water in clays make some of the pore spaces inaccessible to a permeating liquid and, therefore calculated hydraulic conductivity values exceed the actual values by several orders of magnitude. On the other hand, alterations of the characteristics of the electrolyte-soil system changes the double layer characteristics and therefore the interparticle forces. This, in turn, has the potential to alter the soil microstructure, causing the specific permeability term, Cd^2, to change.

The empirical equations suffer from not accounting for interactions between the clay particles and the permeating liquid. Therefore, any application of these equations in determining leachate permeability through clay liners should be viewed with caution.

2.3.2.2 Laboratory Measurement

Because of the increased concern about long term pollution effects associated with sanitary landfills, compacted clay soil liners can no longer be routinely treated as impervious barriers. The bulk permeability across such a liner should be accurately determined. The general consensus is to determine the hydraulic conductivity, k, by laboratory permeability tests and use Eq. 2.1 to calculate the specific discharge of leachate across the compacted clay liner.

Olson and Daniel (1981), provided a good overview of methods available for hydraulic conductivity determination of cohesive soils in the laboratory, and outlined the problems associated with these methods.

In leachate migration studies, it may be useful to measure the hydraulic conductivities under the following two conditions:

- saturated flow where all the pore spaces are filled with leachate,
- unsaturated flow where both leachate and air are

present in the pore spaces.

In a saturated soil, both the moisture content and suction are constant and hydraulic conductivity need only be determined for one direction. The standard test procedure is to use a permeability cell and perform either a constant head or a falling head permeability test.

Methods available for laboratory measurement of the hydraulic conductivity of partially saturated soils are similar to those with fully saturated soils, but the measurement problems are generally more severe (Olson and Daniel, 1981). They identified several sources of error in testing remolded compacted soil samples (see Table 2.3), and some of them (likely to be important for the current study) are:

- errors in sample preparation (non-representative of the field condition),
- alterations in clay chemistry (use of a permeant with no chemical resemblance to leachate),
- growth of microorganisms (a potentially large factor if leachate is used as the permeant).

Dunn and Mitchell (1984) noted the existence of variability in every aspect of commonly adopted permeability testing practice, from sample recovery and preparation to reduction of data. Therefore, before using laboratory determined hydraulic conductivity results to evaluate potential clay liner materials, allowance must be made for these errors. In particular, the actual characteristics of

TABLE 2.3

POTENTIAL ERRORS IN LABORATORY PERMEABILITY TESTS (MODIFIED FROM, OLSON AND DANIEL, 1981)

SOURCE OF ERROR	PUBLISHED DATA (measured k / actual k)
Errors in sample preparation	greater than 1
Use of Distilled water as the permeant	5/1000 to 1/10
Air in sample	1/10 to 1/2
Use of Excessive Hydraulic Gradient	less than 1 to 5
Ignoring volume change due to stress change	1 to 20
Use of wrong temperature	1/2 to 1 1/2

the compacted soil in the full scale liner can be quite different than those used in the laboratory.

Dunn (1983) recognized these limitations and suggested that laboratory tests may be most useful for evaluating:

- field compaction techniques with respect to changes in hydraulic conductivity,
- variation in hydraulic conductivity with dry density and moisture content,
- comparative effects of waste permeants.

In addition to the direct permeability measurement techniques, hydraulic conductivity determinations, using soil consolidation results, have been reported in the literature. Taylor (1948) reported Terzaghi's method of calculating hydraulic conductivity from the results of a one dimensional consolidation testing procedure.

The proposed equation is:

$$k = (T_v \gamma_w L^2 m_v)/t \qquad 2.5$$

Where,

T_v = Terzaghi's time factor,

t = time required for a certain degree of consolidation,

L = drainage path length,

γ_w = density of water,

m_v = coefficient of volume compressibility (de/dp),

de = change in void ratio,

dp = increase in pressure.

The hydraulic conductivity values obtained by calculations are generally lower than the directly measured values (Taylor, 1948; Mesri and Olson, 1971a; Olson and Daniel, 1981).

2.3.2.3 Interpretation of Laboratory Results

Dunn (1983) reviewed the processes involved in subsurface migration of contaminants from a waste disposal site. Processes thought to control the migration phenomenon in all soils are presented in Table 2.4. His review and the subsequent laboratory study revealed the following:

- generally, the hydraulic conductivity of a soil controls the fluid flow velocity (Darcy's law),
- the advection or the bulk motion of flowing fluid normally dominates migration when fluid flow velocity is higher,
- Darcy's law is generally applicable where advection dominates contaminant migration,
- in fine grained soils, molecular diffusion is the dominant mode of contaminant migration, exceeding the net effect of advective flow,
- in fine grained soils, other physicochemical interactions play a big role in determining the extent of contaminant migration.

Under normal conditions, laboratory determined hydraulic conductivity values are generally known to underestimate field values by factors of 10 to 1000. These higher field values are attributed to the occurrence of

TABLE 2.4

Physical and Chemical Processes in
Subsurface Leachate Migration
(Modified from, Dunn 1983)

---

a) PHYSICAL PROCESSES

- Mechanical dispersion
- Molecular diffusion
- Advection

b) CHEMICAL AND BIOLOGICAL PROCESSES

- Adsorption/desorption
- Precipitation/dissolution
- Oxidation/reduction
- Acid/base
- Complexation and chelation
- Biological activity

cracks and fissures in the field soils. Differences of soil structure between lab samples and compacted soil materials, which are exposed to a different environment, may also contribute to this difference.

2.3.2.4 Sample Preparation Methods

A clay liner in the field is constructed using kneading compaction techniques (sheep's foot rollers) at a wet-of-optimum water content. This construction technique is known to produce a dispersed soil structure. Ideally, a sample preparation technique to obtain a similar structure would be desirable for use in the laboratory.

However, the knowledge that the laboratory determined hydraulic conductivity values are always underestimating the field values, seems to imply that the assumption of a completely dispersed fabric is incorrect. The actual nature of the field fabric is not known beforehand. Compactive effort used in the field can vary, creating a soil fabric that is neither fully dispersed nor flocculated.

Dunn (1983) found that the static compaction methods used in the laboratory can produce samples with a higher hydraulic conductivity which is closer to the field values. However, correlations between laboratory compaction and field compaction methods are not yet available.

Samarasinghe et al., (1982) used an artificial sedimentation technique to obtain normally consolidated samples. However, the fabric of such a sample may also differ from the field situation.

2.3.2.5 Permeants

Commonly, the hydraulic conductivity of a clay liner has been determined using distilled water as a permeant (Fuller and Warrick, 1985). However, leaching a sample with distilled water may cause expansion of the diffuse cloud of adsorbed cations around clay particles and reduce hydraulic conductivity. Also, in some cases, leaching could lead to particle migration. Therefore, some studies have used a permeant of similar chemistry to that expected in the field. Agronomists have tried to duplicate a natural pore fluid using 0.01N $CaSO_4$ (Klute, 1972). Environmental engineers, simulating landfill leachates, also used the same permeant (Anderson, 1982).

Fabric alterations that would occur in response to the permeant, necessitate the use of actual leachate or a liquid which chemically resembles leachate, as the permeant.

2.3.2.6 Sample Saturation

Dunn (1983), in reviewing permeability studies done by Green _et al._, (1979) and Anderson and Brown, (1981), noted the apparent lack of techniques for saturation of the test samples in the procedures described. Dunn suggested that the variability of degree of saturation from one sample to another may have resulted in errors in the hydraulic conductivity measurements.

2.3.2.7 Duration of Experiments

Several investigators have reported on changes in the measured hydraulic conductivity as a function of time. Allison (1947) had recognized the natural clogging of soil pores by microorganisms and products of microbial growth as a factor. Investigators tried adding disinfectants to the permeants to inhibit microbial growth to simulate field conditions, where they judged microbial growth to be less likely.

However, in landfill situations, micro-organisms are just as likely to plug up the pores of a liner as they are a lab specimen. Therefore, this factor is probably relevant and should not be overlooked in laboratory tests.

Acids and bases possess the ability to dissolve some components of the soil. Hydraulic conductivity changes, caused by pore blockage with smaller particles produced by dissolution of silica, were reported by many observers (Malcolm et al., 1975).

In addition, there have been observations of initial hydraulic conductivity decreases because of precipitates formed as a result of reactions between permeating liquid and clay. For example, precipitation of $CaSO_4$ was found to explain the maintenance of low permeability for over two weeks, when silty sandy clay soil of medium plasticity (CL) was permeated with liquids having a pH of 1.5 to 2.0 (Gorden & Forrest, 1981). Crim et al., (1979) permeated clay samples initially saturated with a uranium tailings effluent having a pH of 1. They observed a sharp decrease in hydraulic conductivity and attributed this to the precipitation of insoluble salts. Gee et al. (1980) and Schubert et al. (1984), also observed reductions in hydraulic conductivity over time, for the same reason.

Dunn (1983) proposed that, in these cases, hydraulic conductivity starts increasing when the soil buffering capacity is depleted and the precipitates start to redissolve. He also noted that tests over a year in length may not have been long enough to establish the long term effects of permeation with acid liquids.

2.3.3 Factors Affecting Hydraulic Conductivity of Clay Liners

The hydraulic conductivity of the liner depends primarily on the properties of both the porous medium and the permeant. It varies with many factors, including soil density, compaction water content, degree of saturation, void ratio, soil composition, soil structure and permeant characteristics (Wang and Huang, 1984).

2.3.3.1 Factors Related to the Characteristics of the Soil

Hydraulic conductivity is very much affected by the changes in soil structure. The term "structure" is used herein to account for both the soil fabric and its stability. Soil fabric refers to the arrangement of particles, particle groups and pore spaces of a soil. Two soils can have the same fabric but still exhibit different properties, if the forces between particles and particle groups, which govern the stability of the fabric, are not the same.

Theoretical equations, based on capillary tube models for fluid flow through soils, indicate that the flow rate depends on the fourth power of the pore radius. Thus, a soil fabric with a high proportion of large pores will be significantly more permeable than those with smaller pores. The smaller the particle size, the lower the pore size, and the less permeable the soil is. Therefore, the percentage of clay size particles in the soil, particle size distribution and types of clay minerals present in the clay fraction of the soil has a tremendous influence on hydraulic conductivity.

The montmorillonites have the smallest particle size and the kaolinites the largest. Mesri and Olson (1971a), evaluating the hydraulic conductivity of illite, kaolinite and montmorillonite clays in the sodium form, found kaolinite to be about 200,000 times more permeable than montmorillonite at the same void ratio. Illite, which has a smaller particle size than kaolinite but larger than montmorillonite, was about 200 times more permeable than montmorillonite.

Olson (1960) experimenting with kaolinite, illite and Boston blue clay, concluded that pore size distribution of a clay soil was the most important variable affecting hydraulic conductivity.

The clay soil, when compacted to a liner, develops a particular structure in response to the state of applied stresses and the characteristics of the compacting fluid. A flocculated structure, with a more random particle orientation (open), exhibits a larger hydraulic conductivity, whereas, a dispersed structure, with parallel particle arrangements, exhibits a lower hydraulic conductivity.

Mitchell et al., (1965) presented hydraulic conductivity results for samples at the same water content and density but prepared using different compactive effort. The findings (Figure 2.2) show that increased compactive effort increases the particle dispersion. Further results showed the effect of compaction water content on permeability. Higher water contents (wet-of-optimum), produced a dispersed structure depending on the compaction method, whereas low water contents produced flocculated structures.

Mitchell (1955) suggested that the higher shear strains induced with kneading, rather than impact, vibratory or static compaction methods, would produce a dispersed structure when used to compact soil at a wet-of-optimum water content.

The role played by density in determining the hydraulic conductivity was minor, relative to the effect of molding water content and structure (Mitchell et al., 1965).

Mitchell (1976) reviewed the influence of exchangeable cations on various soil properties. Index properties of both montmorillonite and kaolinite clay minerals are affected by the presence of electrolyte in the soil water system (Table 2.5). The effect is very much evident with montmorillonites (bentonite). These effects are caused by a change in the double layer thickness around clay particles (montmorillonites exhibit a larger double layer) and subsequent changes in interparticle forces. Similar situations are also known to cause changes in hydraulic conductivity of soils containing higher percentages of clay mineral montmorillonite (McNeal and Coleman, 1966).

The presence of sodium or potassium ions in the exchange complex of the soil will decrease the hydraulic conductivity. The exchange of sodium or potassium ions by multivalent cations, such as calcium and magnesium, decreases the double layer thickness with resulting tendency towards aggregation and flocculation. These processes increase hydraulic conductivity (Yong and Warkentin, 1969). However, in natural clay soils, with varying percentages of clay and silt particles, ion exchange effects on double layer thickness may be masked by the presence of other particles. The resulting changes in hydraulic conductivity may not be appreciable.

The hydraulic conductivity of clay soil is greatly influenced by the organic and inorganic molecules in the permeants. The concentration and type of cations present

TABLE 2.5

EFFECT OF PORE FLUID ON INDEX PROPERTIES
(modified from Chattopadhyay, 1972)

SOIL MATERIAL	SALINITY AS NaCl (g/liter)	INDEX PROPERTIES		
		Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
Commercial Bentonite	0	591	87	504
	5	417	75	342
	10	343	68	275
	15	279	64	215
	20	256	59	197
Kaolinite	0	72	44	28
	5	68	42	26
	10	66	40	26
	15	66	40	28
	20	67	39	28

(Quirk and Schofield, 1955; Morrison, 1981), the dielectric constant (Green et al., 1981), pH and temperature of the permeants are all known to alter the permeability across compacted clay soils.

Alterations in the clay soil-water system characteristics during permeation may change the interparticle stresses between soil particles and cause a change in the pore size distribution (Lambe and Whitman, 1969). Creation of larger pore channels (as in the case of a flocculated soil structure) within the soil matrix causes hydraulic conductivity, k, to be increased.

The effects of various constituents found in leachates, both organic and inorganic, on clay soil permeability have been studied in great detail by soil scientists, soil engineers and environmental engineers. A detailed review of some of the reported findings are included in section 3.1.

2.4 USEFULNESS OF LABORATORY DETERMINED k VALUES

Proposed US EPA regulations (1978) for disposal of hazardous wastes originally relied on the permeability (or hydraulic conductivity) of a clay soil liner as the primary criterion for preventing the movement of toxic leachates into the ground water (Brown and Anderson, 1980). However, reliance on hydraulic conductivity, which can be strongly affected and radically changed by clay leachate interactions, is open to question. Even the lower concentrations of inorganic and organic ions found in MSW leachates have the potential to alter the structure and volume-change properties of the soil, thereby influencing the permeability characteristics of a clay liner.

Use of hydraulic conductivity as the sole parameter for selecting a soil liner material should not be encouraged for several reasons.

First, the limitations in hydraulic conductivity measuring procedures make these laboratory determined values unreliable. Various additional factors contributing to the unreliability of these values include:

- unrepresentative samples (section 2.3.3.1),
- permeants (section 2.3.3.2),
- duration of experiment (section 2.3.3.4),
- need to use elevated hydraulic gradients,
- failure to consider the variation with time of leachate characteristics.

Because of practical and economic considerations, laboratory permeability tests are not run long enough to study possible long term effects. Since the long term integrity-of the liner is an important consideration, the tests should be run for a sufficient length of time to simulate the long term flow on a basis of total fluid volume. Standard laboratory tests are run for a short duration at high hydraulic heads, expecting to simulate the long term flow on the basis of total fluid volume. But, the elevated hydraulic gradients are known to change the flow characteristics by altering the soil fabric.

Even if a representatve permeant, closely duplicating the leachate characteristics, is used in the tests, it may not fully duplicate the permeant chemistry in the field, which may change with time.

Design and performance of an experimental method, which is devoid of all the deficiencies of the standard permeability testing procedure, is most difficult and very expensive, and may not completely evaluate the real situation in the field. Ultimately, elaborate field testing of the inplace liner may prove necessary to supplement laboratory findings.

Haxo (1981) used landfill simulators to duplicate the landfill situation. Matrecon Inc. (1980) discussed landfill simulators used to evaluate polymeric membrane liner materials exposed to sanitary landfill leachates. These could be adapted to evaluate clay liners and modified to measure the hydraulic conductivity. However, the high expenditure involved in constructing simulators and performing these simulation studies, is a major drawback. Even then, these highly expensive studies fail to account for a situation most likely to occur in cold climates. The permeability specification fails entirely in situations where the clay liners are found to have cracks even before beginning of landfill operation. Hydraulic conductivity, measured as accurately as possible using the best methods, will not apply where breakthrough and bulk transport may dominate the mode of leachate migration.

Breakthrough and bulk transport of leachate can be initiated in two ways:

- Organic solvent breakthrough has been observed by Green et al., (1981). Ridley and Bewtra (1983) reported liquid breakthrough when clay soils are permeated with high concentrated brine. Displacement of water molecules from the clay micelles, by the inorganic and organic ions, could cause soil shrinkage and subsequent cracking in clay soils.
- A clay liner can develop cracks due to drying or freezing action, once installed at the site and exposed to the weather.

Subsequent breakthrough and bulk transport can occur if the produced leachate further deteriorates the liner by widening the opened cracks and allowing soil erosion to take place.

MSW leachate is an aqueous solution with varying amounts of organic and inorganic ions. Schubert et al. (1984), performed direct permeability tests on natural and recompacted soil samples, obtained from a glacial clay liner, to determine whether any changes in hydraulic conductivity could occur upon exposure to a dilute highly acidic leachate. The low plastic clay soils (PI less than 15) exhibited no significant change in k. Based on these laboratory observations, they concluded that highly concentrated organic acids and low concentration of organic and inorganic cations present in leachate, do not influence the physical characteristics of the clay liner. Since the field performance of this liner was not provided by them, the validity of their deductions could not be verified.

Observations in the literature suggest that an estimation of the field hydraulic conductivity of a liner, based only on conventional permeability testing in the laboratory, without consideration of all the relevant physical and physico chemical factors when the liner is in contact with leachate, could certainly be misleading.

3. VOLUME CHANGE IN CLAY SOILS AND COMPACTED CLAY LINERS

Soils undergo volume changes in several different ways: swelling caused by ingress of water, rebound resulting from release of load, settlement caused by load application, or shrinkage resulting from water loss (Subba Rao & Satyadas, 1986). To be effective in retarding migration and confining the leachate within a set boundary, clay soil materials must maintain their micro and macro properties within the lifetime of the landfill.

The reactions between clay soils and leachates are known to affect soil characteristics, such as volume change behavior and permeability. A direct consequence of volume change, is the development of cracks within the soil and subsequent alteration of the bulk permeability. Accounts of bulk permeability changes because of crack formation, mainly resulting from thermal and chemical dessication, are readily found in the soil science literature (Thomas and Phillips, 1977, Beven and Germann, 1982, Bouma, 1984).

Dessication or removal of water from the soil matrix, due to external influences, can be manifested by temperature changes. Thermal dessication and subsequent crack formation, resulting from drying and wetting cycles (Macey, 1942) or freeze-thaw action (Hamilton, 1966) are known to be important in semi-arid cold climates.

As well, volume change caused by changes in the physical/chemical characteristics of the pore water, as a result of MSW leachate reactions with the soil matrix must be considered. These leachate-clay soil interactions are the primary focus of this thesis.

3.1 CLAY-LEACHATE INTERACTIONS

Table 2.4 presented the most important reactions that have to be considered in order to understand the movement of leachates from waste disposal facilities. Chemical reactions, important in a landfill situation, are expanded further and presented in Table 3.1.

The soil matrix below the landfill, which may or may not include a compacted clay liner, is capable of attenuating pollutants and retarding their progress toward groundwater sources. The compacted clay liner, contributes considerably to these two processes by promoting attenuation and acting as a barrier to leachate.

Table 3.1 outlines clay soil interactions with leachate and soil capabilities to alter the characteristics of the leachate. On the other hand, these reactions also indicate the capability of landfill leachate to change the basic characteristics of a clay liner. Biological, physical and chemical reactions are capable of altering the basic soil structure and corresponding volume change, permeability and strength characteristics (the three basic engineering properties) of a soil.

The interactions between clay soils and leachate constituents can be categorized as :

- attenuation reactions, that can change the characteristics of the landfill leachate
- reactions capable of altering the volume change, strength and permeability characteristics of the liner.

TABLE 3.1

SOIL - LEACHATE INTERACTIONS

---

a) REACTIONS THAT CAN CHANGE THE LEACHATE QUALITY

- BIOLOGICAL ACTIVITY
- HYDRODYNAMIC DISPERSION
- ADSORPTION AND ION EXCHANGE PROCESSES

b) REACTIONS THAT CAN CHANGE THE FUNCTIONAL PROPERTIES OF THE LINER

- REDUCE BULK PERMEABILITY
  - BIOLOGICAL ACTIVITIES
  - PRECIPITATION REACTIONS
- INCREASE BULK PERMEABILITY
  - ADSORPTION AND ION EXCHANGE REACTIONS
    - with organics
    - with inorganics

    (resulting in structural collapse, clay dessication and synaerisis shrinkage)

Research activities have been directed towards understanding the soil attenuation characteristics. Movements of leachate plumes below landfills have been monitored and various mathematical techniques have been proposed to understand the dispersion/dilution processes below the landfill. Laboratory work has been concentrated in developing chemical breakthrough curves for specific solutes for various soil materials. Determination of retardation factors, for specific solutes and waste permeant advancement monitoring, allowed the assessment of the degree of chemical degradation within a porous medium in the laboratory (Haji-Djafari and Wright, 1983).

Similar techniques have been adopted in studying the biological degradation and adsorptive retention of various organic and biological (microbial) pollutants, within a porous soil matrix. (Pohland and Engelbrecht, 1976)

Recently, bulk permeability changes, caused by the permeation with waste components, have generated attention. Anderson et al., (1981, 1982), D'Appolonia (1980), Gordon and Forrest (1981), Green et al., (1979, 1981), Haji-Djafari and Wright (1983) evaluated the hydraulic conductivity, and observed a marked change in this soil parameter when permeated with various waste components. They used primarily pure organic solvents in their experiments. Ridley and Bewtra (1983) used brine as the waste permeant and saw a large increase in hydraulic conductivity.

In all these studies, time dependent variation of the hydraulic conductivity because of chemical solvents (mostly pure organic solvents) was the prime target. Recently, Daniel and Liljestrand (1984) performed experiments to determine whether dilute organic chemicals present in landfill leachates are likely to cause significant increases in the hydraulic conductivity of compacted clay soils. Permeability tests showed that none of the five leachates (one of them being a leachate from a solid-waste landfill, the other four containing mainly low concentrations of organic compounds), tend to cause increases in laboratory measured hydraulic conductivity. However, the MSW leachate used by Daniel and Liljestrand (1984) contained a very low total dissolved solid content (1043 mg/l, well below the values cited in Table 2.1), indicating a low inorganic cation content.

Olson and Daniel (1981) have noted the major limitations in attempting to evaluate field hydraulic conductivity in the laboratory. Some researchers have attempted to refine the hydraulic conductivity determination procedures, to account for the waste permeant-soil interactions. Dunn (1983), reviewed the limitations and current methodology in determining hydraulic conductivity for waste permeants and proposed a new procedure for laboratory measurement.

The basic premise of the foregoing research was:

- waste permeants (from industrial, municipal landfills, ponds, etc.) interact with soil materials and may drastically change their characteristics,

- k, the hydraulic conductivity determined following standard procedures (using, distilled water or 0.01N $CaSO_4$ as the permeant in an inflexible permeater), usually underestimates the actual k caused by waste permeants,
- k which is dependent on the micro characteristics of the soil (basically the pore structure voids) can be changed whenever the waste permeants change these micro characteristics.

Therefore, to understand changes in k as a means to understanding the macro permeability of the liner (bulk permeability is a characteristic of the liner, whereas k, the hydraulic conductivity, is a characteristic of the soil material), the interaction between the leachate and soil should be carefully studied. As an example, a close examination of the governing equations of diffuse double layer theory (Arnold, 1978) would predict the observed effects of organic permeants on hydraulic conductivity.

According to Haxo et al., (1983), MSW leachates, containing various organic and inorganic ions, are not known to cause drastic changes in ~~physical~~ characteristics of various clay liner materials. Daniel and Liljestrand (1984) did not observe increased permeability with dilute organic solutions or a dilute MSW leachate. However, research attention has generally been focussed on high-strength waste permeant-soil clay interactions (including brine, pure organic solutions) and not much attention has been directed to MSW leachate and clay soil interactions.

However, numerous incidents (Quigley et al., 1984), where MSW landfills have polluted underlying groundwater sources, suggest that MSW leachate-clay soil interactions and their implications in a MSW landfill situation need to be carefully evaluated.

3.1.1 Clay Soil and Inorganics

Soils cannot be treated as inert engineering materials with fixed properties. Soil properties and behavior can be changed by chemical attack with an active permeant.

The solid phase of a given soil is primarily comprised of crystalline clay and non-clay minerals, non-crystalline clay material, organic matter and precipitated salts. Although the amount of clay mineral is small relative to non-clay materials, the interaction between a permeant and soil particles is dominated by the clay mineral fraction. Therefore, the amount, type and properties of the clay particles present are of major concern (see Appendix B1 for a general description of the properties of common clay minerals).

Mitchell (1986) discussed the importance of understanding local chemical and mineralogical factors influencing soil properties and behavior, by an illustrative example. He reported the delayed failure of lime-stabilized pavement bases in Las Vegas, Nevada, and put forward a hypothesis to explain the unexpected swelling observed. A series of chemical reactions between the lime treated swelling soil (dominant clay mineral being montmorillonite) and the surrounding liquid environment, led to the formation of very expansive ettringite and thaumarite (Mehta and Hu, 1978). These were thought to be responsible for the subsequent surface heave (occurred almost 2 years after construction was completed), and cracking. Although water immersion following compaction of lime and cemented sulfate-bearing salts (as in this case), are reported to lead to expansion and cracking (Sherwood, 1962), a limited understanding of the process and lack of prior experience in such situations, prevented engineers from anticipating this phenomenon.

This experience showed the importance of studying possible soil behavior with respect to its environment, over a long period of time, especially in situations where little or no prior experience is available.

Soils, especially the clay fraction, undergo changes in their physico-chemical properties as a result of interactions with the inorganic ions in the leachate. The reactions, summarized in Table 3.1, were discussed in detail by Pohland and Engelbrecht (1976), with respect to their contribution in removing pollutants from leachate. Pollutant attenuation, or a soil property/behavior change, occurs as a result of a combination of these reactions. Therefore, to understand or explain a soil property change, their combined effect should be examined. Specifically, ion exchange reactions seem to contribute significantly to understanding the effect of various inorganic ions on soil volume change behavior.

3.1.1.1 Important Properties of Clay Soils

The interaction between leachate and clay liner material is dominated by the clay mineral fraction, because of the: large surface area, surface charge and ion exchange properties.

In general, the surfaces of clay minerals are negatively charged, as a result of charge contributions associated with the crystal structure of clay minerals (Mott, 1981).

There are two components of negative charge; a constant or permanent charge and a variable pH-dependent charge.

Charge on a clay particle is due to:

- isomorphous substitution,
    - substitution of an ion by another of nearly equal size but of lower valence, leaving a net negative charge in the lattice. Main substitutions found are aluminum for silicon in the silicon sheet, magnesium, iron or lithium for aluminum in the aluminum sheet. These account for most of the charge in 2:1 minerals (eg. montmorillonite), but only a minor part in the 1:1 kaolinites.

- Broken bonds,
  - unsatisfied valence charges at the edges of the particles. Charge caused by this increases with a decrease in particle size, because the proportion of edge area to total area is increased (common in Kaolin).
- soil organic matter,
  - the negative charge on organic matter arises from ionization of hydrogen from carboxyl and hydroxyl groups, and accounts for about one third of the exchange capacity of a surface soil.

The permanent negative charge, arising from the isomorphous substitutions is constant, at least over the pH range from 2.5 to 10.0 (Talibudeen, 1981). At pH values less than 2.5, this charge decreases, because the mineral breaks up slowly at crystal surfaces and edges. The contribution to negative charge, from soil organic matter, increases with increasing pH.

Charge caused by broken bonds would also depend on the pH of the system (Stumm and Morgan, 1981). The broken bonds are known to attract hydrogen or hydroxyl ions from the pore water (Yong and Warkentin, 1975). The ease with which the hydrogen (existing as H_3O^+ in solution) can be exchanged increases as the pH of the pore water increases. Therefore, the charge caused by broken bonds increases as pH increases.

In kaolin, at low pH values, broken edges may become positively charged by attracting extra H_3O^+ ions. This explains the flocculation behavior of kaolinites at low pH values (attraction of positive and negative charges).

In general, positive charges in clay particles originate from the specific adsorption of protons (H^+ ions) on suitable sites (oxide/hydroxide surfaces). Therefore, the magnitude of this charge depends more critically on the ionic strength of the solution, than does the negative charge (Bache, 1977).

When pH of the solution is between 4.0 and 7.0, there are usually some positive charges to which anions can be electrostatically attracted (Mott, 1981). The positive charge is strongly pH dependent and is substantially neutralized by OH^- at pH 7.0 (Bache, 1977).

3.1.1.2 Ion Exchange

The mineralogy of clays, with respect to their structural configuration and exchange capacities, is well documented by Grim (1968). A summary of important aspects of major factors contributing to the properties of clay minerals are summarized in Appendix B1.

As noted above, the solid phase particles of the soil often carry a negative surface charge distributed over the basal surfaces. The overall neutrality of the system is then maintained by the presence of an excess of cations, in comparison to the anions present, in close proximity to the solid surface. These cations are linked to the lattice by relatively weak electrostatic forces. They are weakly held on the surfaces and the edges, and in some clays, between the unit cells.

This excess of cations cannot be removed from the system, but it is possible to exchange these cations, hereby called "exchangeable cations", while maintaining the electroneutrality of the system (Bolt and Bruggenwart, 1976). These cations are termed, "exchangeable", because one cation can be readily replaced by another of equal valence, or by two of one-half the valence of the original (Yong and Warkentin, 1975). For example, if a sodium clay is washed with a solution of calcium chloride, each calcium ion would replace two sodium ions. The reaction can be written as:

$$Na_2\text{-clay} + CaCl_2 \leftrightarrow Ca\text{-clay} + 2NaCl$$

This reversible process is called "cation exchange" and is important in understanding fundamental soil behavior. The quantity of exchangeable cations held in this manner is the cation exchange capacity of the soil and is approximately equal to the amount of negative charge.

The predominant exchangeable cations in soils are calcium and magnesium. In general, potassium and sodium are found in smaller amounts. However, the geologic environment and subsequent leaching, determine the dominant cations in a particular soil.

Clays deposited in sea water will have predominantly magnesium and sodium. Calcareous soils contain mostly calcium. Dominant exchangeable ions in acid soils are aluminum and hydrogen.

3.1.1.3 Cation Exchange Capacity of Soils

Cation exchange reactions can be considered as a competition between different cations for the negatively charged surfaces to which they are electrostatically attracted. The distribution of each cation between the surface and the pore solution depends on several factors (Bache, 1977).

These factors include:

- the charge, size and hydration state of the cations,
- the surface charge density of the soil particles,
- the ionic strength of the solution and
- any specific interactions that may occur (eg. size preference for K^+ by some minerals, organic-complexing of some cations).

Cation exchange capacity is, therefore, a measure of the ability of the soil to adsorb cations in such a form that they can be readily exchanged by competing ions in the pore solution. Because of the large quantity of unbalanced substitutions in the lattice structure, montmorillonite exhibits the greatest exchange capacity among clay minerals, generally in the range of 80 to 150 meq/100g (Mitchell, 1976).

Although no evidence of isomorphous substitution exists, kaolinites also possesses a net negative charge and values of cation exchange capacity in the range of 3 to 15 meq/100g. Measurement of low exchange capacities, under low pH conditions, suggest that kaolinite particles are actually charged positively on their edges when in an acid environment (Mitchell, 1976).

Illite or hydrous mica, also displays isomorphous substitution, but records a lower cation exchange than montmorillonite, amounting to 10 to 40 meq/100g (Table 3.2). in Illite, the charge deficiency is located primarily in the silica sheets and is balanced partly by the non-exchangeable K^+ ions located between layers.

The capacity of different soils to exchange cations is also related to their adsorptive properties, particularly with respect to large organic molecules. Therefore, the particle size and the specific surface area of clay minerals become important considerations. Table 3.2 shows that montmorillonite, with the smallest size, displays the largest surface area, followed by illite and kaolinite.

3.1.1.4 Exchange Equations and Ion Selectivity

Exchange reactions, are rapid and the amount of accessible surface for exchange is very high. Once equilibrium has been reached, there exists a relationship between the composition of the exchange complex and that of the soil solution. A number of equations are available to explain this relationship. The proportion of each exhangeable cation to the total exchange capacity, as the outside ion concentration varies, is given by an exchange-equilibrium equation.

TABLE 3.2

CLAY MINERAL CHARACTERISTICS

CLAY MINERAL	SHAPE	SIZE	CATION EXCHANGE CAPACITY (meq/100 g)	SPECIFIC SURFACE (m^2/g)
Kaolinite	hexagonal plate	diameter 0.3 - 0.4 microns thickness 0.05 - 2.0 microns	3 - 15	10 - 20
Montmorillonite	thin plates	diameter 0.1 - 1.0 micron thickness 30 Angstroms	80 - 150	50 - 120 Primary * 700 - 800 Secondary *
Illite	plates	thickness 300 Angstroms	40 - 40	65 - 100

*external surface exclusive of interlayer zones

*surface determined by expanding the lattice, therefore, include interlayer surface area too.

1 Angstrom = 10^{-4} micron = 10^{-10} meter

The distribution between solution and exchange complex of ions of equal valence is governed (Bolt and Bruggenwert, 1976) by:

$$\frac{B_e}{A_e} = k_{a/b} \frac{\{B^+\}}{\{A^+\}} \qquad 3.1$$

where,

A & B are solution cations of the same charge,

{ } denotes activities,

A_e,B_e are concentration of adsorbed (or exchangeable cations) ions,

$k_{a/b}$ = selectivity coefficient (dimensionless)

$k_{a/b}$ is specific for any ion pair and may differ for different clay minerals.

From the experimental evidence, the relative preference for monovalent cations is:

$$Cs^+ > Rb^+ > K^+ \cong NH_4^+ > H^+ > Na^+ > Li^+$$

This indicates that the radius of the hydrated cation (Table 3.3) determines the magnitude of the selectivity coefficient. Given that the solvated radius varies inversely with the crystal radius and that the polarizability of the ion is a function of crystal radius, selectivity is more marked when polarizability and ion charge is high (Bonnen & Souchier, 1982).

In addition, there are specific effects caused by either the ion or the clay mineral surface which influence the replacing power. In most clays, for the common cations,

TABLE 3.3

IONIC RADIUS OF SOME COMMON CATIONS
(adapted from Mitchell, 1976)

CATIONS	NON-HYDRATED RADIUS (Angstroms)	HYDRATED RADIUS (Angstroms)
Na	0.98	5.6 - 7.9
K	1.33	3.8 - 5.32
NH	1.43	5.37
Mg	0.78 - 0.89	10.8
Ca	1.06 - 1.17	9.6
Al	0.45 - 0.79	---
Si	0.31 - 0.39	---

Na^+ and K^+, K^+ is preferred with a selectivity coefficient of about five (Bolt and Bruggenwert, 1976). A similar effect is found for divalent cations, with Ca^{+2} preferred over Mg^{+2} by a factor of about 1.2.

Of special interest is the heterovalent exchange process, particularly the exchange equilibrium between the dominant cations Na^+, K^+ and Ca^{+2}, Mg^{+2}. For the exchange of monovalent sodium (Na) and divalent calcium (Ca), the Gapon equation can be written as (Arnold, 1978):

$$\frac{Na_e}{Ca_e} = K_G \frac{Na}{\sqrt{Ca/2}} \qquad 3.2$$

K_G is the empirically determined Gapon exchange constant, with Na_e and Ca_e being the concentrations of sodium and calcium in milliequivalents, within the exchange complex (or adsorbed from the equilibrating solution).

K_G depends upon specific cation adsorption effects and upon the clay surface, decreasing as the surface density of charge increases. When cation concentrations are expressed in milliequivalent, most of the soils exhibit a Gapon constant, K_G, of 0.017 (Mitchell, 1976).

According to this equation, for a simillar concentration of Na^+ and Ca^{+2} ions (0.02M) in solution, nineteen times as much Ca^{+2} ion is present in exchangeable form (Yong and Warkentin, 1975). This demonstates the valence effect on cation exchange. Important cations can be arranged in a series, in the order of increasing replacing power (Taylor, 1959):

$$Na^{+} < H^{+} < K^{+} < NH_{4}^{+} < Mg^{+2} < Ca^{+2} < Al^{+3}$$

The actual position will also depend upon the type of clay mineral and the ion which is being replaced. As noted earlier, the number of exchangeable ions replaced, depends upon the concentration of ions in the replacing solution. Specifically, a cation of high replacing power can be dislodged by a cation of low replacing power if the latter is present in high enough concentration (Hillel, 1971).

3.1.2 Permeability Changes Caused by Soil-Inorganic Ion Interactions

Wide use of clay-lined, surface storage reservoirs for brine has highlighted the problems associated with brine and clay soil interactions. Various studies including, Yanful (1982), Ridley (1985), and Komex Consultants (1985), demonstrated the effect of brine (about 30% by weight NaCl solution) on various engineering properties of compacted soil, especially the bulk permeability. However, the effect of electrolyte on bulk permeability has been a concern for soil scientists for a very long time and a wealth of information is available in the soil science literature dealing with the effect of relatively weak electrolyte solutions.

Waldron and Constantin (1968), studied both bulk volume and hydraulic conductivity changes which were caused by changing permeant characteristics. They permeated loamy

Californian soils, compacted and constrained under a 0.05 bar axial pressure (equivalent to an overburden pressure on a layer of soil at a depth between 0.3 to 0.6 m), with 1M NaCl, followed by 0.2M NaCl solution, and measured the bulk volume and hydraulic conductivity changes. In certain cases, they reintroduced the 1M NaCl solutions to assess the reversibility of the changes observed. They reported three hydraulic conductivity-specific bulk volume relationships and attempted to interpret them by considering the chemically induced changes on pore size distributions (as opposed to total pore changes). According to them, the size, shape and distribution of macropores, control the hydraulic conductivity. They rejected the hypothesis presented earlier by Lutz and Kemper (1959), that fluid viscosity close to solid surfaces very much controls the flow of an electrolyte through very fine pores such as occurs in pure clays. The loam soils used by Waldron and Constantin (1968) contained micropores within the soil aggregates and macropores external to the aggregates.

Rolfe and Aylmore (1977) studied the passage of various cations (Ca^{+2}, Na^{+}, Cs^{+2}), at a range of concentrations (0.01M to 3.5M), through compacted clay minerals (Willalooka Illite, and Wyoming Bentonite). They observed a marked dependence of hydraulic conductivity on both the nature of the exchangeable cation and electrolyte concentration. They attributed increases in hydraulic conductivity, with high concentrations (Table 3.4) and high valences, to several

TABLE 3.4

EFFECT OF INCREASING SOLUTION CONCENTRATION ON SPECIFIC PERMEABILITY - PRESSURE GRADIENTS

(modified from Rolfe and Aylmore, 1977)

CATION	SOLUTION CONCENTRATION (M)	GRADIENT (10^{-18} mm^2 / 6.89 kPa)
	0.017	1.06
Na	0.12	1.10
	1.03	1.60
	0.011	2.00
Ca	0.682	2.30
	3.50	11.98

mechanisms, the most important being the changes incurred on the ionic distribution around clay platelets.

The following mechanisms were suggested as contributing to increased or decreased hydraulic conductivity:

- alterations in the pore dimensions as a result of changes in swelling forces within the soil matrix,
- variations in the magnitude of the electroviscous retardation of flow (important contribution in clay, according to Lutz and Kemper, 1959),
- changes in the mobility of water molecules associated with exchangeable cations (poorly hydrated and strongly adsorbed cations provide less hindrance to the flow),
- Alterations in the actual viscous behavior of the water structure (as indicated by cation Cs^{+2}).

Fireman (1944) conducted permeability tests on a sandy loam soil, with permeants of varying degrees of electrolyte concentration. The lowest hydraulic conductivity ($2x10^{-7}$ m/s) was observed with water. Values of $1x10^{-6}$ m/s and $6x10^{-5}$ m/s were obtained when permeants were changed to 4500 mg/l sodium chloride and 800 mg/l calcium chloride, respectively.

Similar results have been obtained later by Reeve et al., (1954), Lambe (1954), and Mesri and Olson (1971a). Their combined results showed that, in general, increasing the electrolyte concentration in the pore water would cause an increase in the hydraulic conductivity, so the conductivity is greater for electrolyte solutions than for distilled water. They found that the hydraulic conductivity is lower when the adsorbed (or exchangeable) cations are monovalent rather than divalent.

Mesri and Olson (1971a), calculated hydraulic conductivities from consolidation experiments performed on homoionized clay minerals (illite, kaoline and smectite) with $CaCl_2$ and NaCl of concentrations 0.0001N and 1N, as pore fluids. They noted that a reduction in the electrolyte concentration in the permeant tends to reduce the hydraulic conductivity, but the effect diminishes as the valency of the cation increases and is smaller in sequence from smectite to illite to kaolinite.

Mesri and Olson (1971a) tried to explain these observations based on the effect of electrolyte on dispersion or flocculating behavior of clay minerals. Aggregation or flocculation creates tiny flow channels contributing only a little to the flow, and a small number of relatively large channels through which the main flow seems to occur. Dispersion creates mainly uniform, small channels, resulting in a low fluid flow.

The same explanation, (flocculation increases soil permeability while swelling and dispersion decreases permeability) was suggested for observed hydraulic conductivity changes by Hardcastle and Mitchell (1974), during permeation of artificially sedimented mixtures of Na illite and a ground basalt with 0.05N to 0.6N NaCl, and by

Gorden and Forrest (1981), while testing compacted clay and artificially sedimented muds with alkaline and acid permeants (moderate electrolyte concentrations).

The foregoing research involved the interaction of various soil materials with low to moderate electrolyte concentrations. Hydraulic conductivity change induced by changes in permeant characteristics was caused by a combination of mechanisms manifested by alterations in the physical/chemical forces within the soil-water-electrolyte system. Soil engineers tend to analyze changes in hydraulic conductivity in the context of soil structure characteristics (whether flocculated or dispersed), whereas soil scientists suggest a number of interacting mechanisms to explain similar observations.

Regardless, when it comes to high-strength electrolytes, it appears that bulk permeability changes are controlled by other mechanisms. Shrinkage cracking, which results from volume contraction induced by synaerisis (loss of water due to internal mechanisms), creates gross void volume defects or macropores. Such voids result in large increases in measured hydraulic conductivity, reported as "breakthrough" by Ridley and Bewtra (1983) and Green et al., (1981).

Yanful (1982), reported the observation of synaerisis cracks in clayey soils exposed to brine. His hydraulic conductivity measurements indicated that the hydraulic conductivity of a "fresh water clay", with a brine permeant, is about 100 times greater than when a fresh water permeant is used. Development of synaerisis cracks in the very thin samples (25 mm), compacted to their optimum water content, resulted in the observed high hydraulic conductivities.

Occurrence of synaerisis cracks, due to high electrolyte concentrations, were reported earlier by Rich (1951), White (1961), Burst (1965) and Plummer and Gostin (1981). However, the importance of this phenomenon in compacted soils has not been adequately verified.

Ridley and Bewtra (1983) studied long term exposure to 30% NaCl brine of fine grained soils, high in the clay minerals montmorillonite, illite and kaolinite. Although, engineering properties, including hydraulic conductivity, of montmorillonitic soils were affected severely, brine soaking had little effect on soils rich in illite and kaolinite.

They determined hydraulic conductivity for compacted (at optimum moisture content), 100 mm thick soil samples in a plexiglass permeameter, and observed a "breakthrough phenomenon" with several systems. Because exchangeable cation data was not reported (some idea can be gained by the given soluble salt data), a comprehensive analysis of their results is not feasible. The breakthrough (sudden increase in k) of brine was likely due to the action of synaerisis and subsequent crack formation. Lack of restraint in the vertical direction during brine permeation, would allow either the separation of the soil sample from the rigid permeameter walls and/or formation of tiny cracks within the soil sample itself during the shrinkage process. This was also evident in earlier experiments performed by Green _et al._ (1981), where "breakthrough" of organic permeant was observed with organic permeants on compacted samples, with only a small vertical restraint.

"Breakthrough" or the sudden increase in permeability, is the result of the formation of larger than normal flow channels within soil sample. It can only be explained by considering the volume shrinkage phenomenon in soils.

Recently, in order to study the performance of a clay till liner in a brine pond in Alberta, Komex Consultants (1985) performed several falling-head permeability tests on undisturbed, remoulded, contaminated (with brine) and uncontaminated soil samples, with brine solutions. They compared the measured hydraulic conductivities with the recorded field values and found wide variations between them. This is consistent with other reported case studies (Olson and Daniel, 1981 and Daniel, 1984). They also reported an increase in k of about one order of magnitude for brine (compared with tap water) on samples moulded with tap water.

In addition, their permeability results indicated that pre-conditioning of soil, through brine soaking, does not have an appreciable influence on the compacted permeability. However, they concluded that brine soaking may aid in limiting crack formation in the field and in reducing the field bulk permeability. They contended that their results confirm the difficulty of measuring absolute field permeabilities (which are governed by soil volume changes, in this case) by performing laboratory permeability tests on undisturbed or recompacted samples.

3.1.2.1 Analysis of Case Studies

Comprehensive records of clay soil liner behavior in MSW landfill situations is not available. However, Komex Consultants (1985) and Daniel (1984) presented case records to illustrate the effect of concentrated salt brines on compacted clayey materials.

Komex Consultants (1985), presented results obtained by EBA Engineering consultants Ltd., during their 1978 investigation of IOL (Imperial Oil Limited) Brine Pond, in Redwater, Alberta. Effective field permeability of the pond liner (prepared from clayey silt, which had an average gradation of about 2% gravel, 34% sand, 43% silt and 21% clay with plasticity index 12% and liquid limit 28%) was about $2.5x10^{-8}$ m/s. This was significantly higher than the laboratory determined hydraulic conductivities for undisturbed samples, which was in the range of $3x10^{-11}$ m/s to $3x10^{-10}$ m/s. Occurrence of volume defects (cracks, slickensides, fissures and roots) in the field were believed responsible for this difference.

Komex Consultants (1985), reported studies done by Hudec (1978) on the IOL Brine Pond, where leakage of brine has been detected and the pond was considered unserviceable. He performed permeability tests on 30 mm and 60 mm thick samples remoulded from IOL till material, and found that permeabilty for brine was approximately 5 times greater than for the same sample permeated with fresh water.

He attributed the observed hydraulic conductivity increases to:

- shrinkage of the structure during decreases in the double layer thickness around clay particles,
- a re-orientation of clay particles from a dispersed to a flocculated condition,
- osmotic pressure differences leading to a transfer of fresh water from the clay micelle to brine solutions,
- hot brine decreasing the double layer thickness and leading to shrinkage of the material.

Komex Consultants (1985) also reported the case of the Chevron Brine Pond, in Redwater, Alberta, where exposed pond sideslopes and the pond bottom displayed significant shrinkage and cracking because of exposure to a highly concentrated brine. In spite of significant cracking near the surface, the presence of clay & clayey silt material beneath the cracking zone tended to limit brine seepage.

3.1.3 Clay and Organics

Although interaction of clays and organic compounds are well documented in the literature (Grim, 1968 and Theng, 1974) this vast knowledge has not yet been applied in designing most waste disposal facilities. Again, most of the research has been directed towards clay's capability to extract unwanted ionic species from the leachate (Pohland and Engelbrecht, 1976).

Although the potential for attenuation of organic molecules by clay soils could be explained broadly in terms of ion exchange and adsorption, a full understanding of the extent of these attentuation mechanisms involves identification of more complex reactions.

In recent years, more research has been focused on the rate of movement of organic solvents through soils, and interactions that are detrimental to the basic properties of the soil material (Kingsbury and Truesdale, 1985). Although most researchers performed direct permeability tests to determine hydraulic conductivity of the soil material, their results and discussions provide a wealth of information on the fundamental reaction that may be responsible for changing the hydraulic conductivity of waste permeated soils. A comprehensive review of past research involving permeability testing with organic chemicals is presented by Kingsbury and Truesdale (1985).

Macey (1942) investigated clay permeability to polar and non-polar liquids. His results showed the rate of flow for benzene to be 100,000 to 1,000,000 times greater than for water in the same clay. He considered the most important cause for the lower permeability of water to be the anomolous viscosity of water near the clay surface.

Michaels and Lin (1954) demonstrated the dependence of hydraulic conductivity on certain characteristics of the permeants. They found that the hydraulic conductivity decreases with increasing polarity of the permeate. They judged the degree of soil dispersion to be the most important factor controlling hydraulic conductivity. Mesri and Olson (1971 b) also evaluated the effect of organic solvents (polar and non-polar) on hydraulic conductivity. Values derived by consolidating homionic kaolinite, illite and montmorillonite samples showed good agreement with results presented by Michaels and Lin (1954).

Green _et al._, (1981) attempted to relate, empirically, the hydraulic conductivity, k, to the dielectric constant of the solvent and the bulk density of the soil. Their results suggest that, the greater the dielectric constant, the greater will be the value of k. In fact, the water (largest dielectric constant) permeability exceeded the k values of all the tested organic solvents. These results are in direct conflict with the results obtained by Macey (1942) and Michaels and Lin (1954). However, Green _et al._, (1981) correctly noted the invalidity of permeability results in situations where solvents with low dielectric constant (eg. CCl_4) shrink the clay soils and cause breakthrough and bulk transport of solvents to occur.

Later, they presented data on bulk swelling/shrinking of 3 clay soils exposed to organic molecules, under soil moisture conditions approximating those found in hazardous waste disposal sites (Green et al., 1983). Using a consolidometer to measure these volume changes, (which would be expected from double layer theory), they obtained greatest swelling for water, which had the greatest dielectric constant, and highest swelling potential. Shrinkage, due to water loss in the clay soil, was observed for both xylene and CCl_4 which were non-polar solvents with very low dielectric constants.

Anderson et al., (1982) studied organic solvent permeability and obtained results more in agreement with those of earlier researchers. When tested with standard aqueous leachate (0.01N $CaSO_4$), four compacted clay soils showed hydraulic conductivities lower than 1 x 10^{-11} m/s. The soils showed higher hydraulic conductivities when permeated with neutral non-polar fluids (xylene and heptane). Alteration of soil fabric was given as the cause of the approximately two orders of magnitude increase in hydraulic conductivity. Weak organic acids (acetic acid) are thought to change the hydraulic conductivity by dissolving soil particles, followed by piping of the particle fragments through the soil.

However, Dunn (1983) pointed out that in hydraulic conductivity experiments performed by Green et al., (1981) and Anderson et al., (1982), no provision had been made for saturation of the samples. As noted in Section 2.3.2.6, use of partly saturated soil samples in permeability testing contributes to the wide variability of hydraulic conductivity results. Therefore, the possibility that the degree of saturation may vary from one test to another makes their results highly unreliable. Daniel and Liljestrand (1984) also criticized permeability tests performed by Anderson et al. (1982) for several reasons, including the use of very large hydraulic gradients and rigid wall permeameters.

In general, neutral polar organics such as alcohols, ketones, and amines are known to form interlayer complexes with clays and to cause volume increases in the clay lattice (Theng, 1974). On the other hand, non-polar low-dielectric solvents (aliphatic and aromatic hydrocarbons) tend to displace water from clay aggregates; this, in turn may cause volume shrinkage (Anderson and Brown, 1981).

Therefore, it is highly likely from the test procedures and results reported by many investigators (Macey, 1942; Michaels and Lin, 1954; Brown and Anderson, 1980; Anderson *et al.*, 1982; Green *et al.*, 1981, 1983) that the large increases in hydraulic conductivity, following permeation with low-dielectric organic solvents (CCl_4, xylene, benzene) were caused by crack formation, following shrinkage, caused by "synaerisis" (Gray, 1984). Synaerisis shrinkage, volume decrease as a result of liquid loss from the clay micelle, can occur following exposure of the clay to certain types of organic fluids, and leachates containing large concentrations of electrolytes. This phenomenon is of particular interest as a mechanism of induced shrinkage cracking in clay liners, and hence will be discussed in detail, Section 3.3.4.3.

Dissolved organic carbon is high in the landfill leachates (Table 2.1). The largest portion of these molecules are organic acids. Even high concentrations of these organics produces only a minute change in the dielectric constant of the leachate, which will not differ significantly from that of water. Therefore, the polar and non-polar organic molecules present at concentrations normally found in the MSWleachate do not contribute to the shrink-swell behaviour, but do take part in the attenuation reactions.

However, organic acids (sometimes take up about 75% of the total organic content) are known to pose some problems (Brown and Anderson, 1980). The solubility of clays in acid varies with the nature and concentration of the acid, acid to clay ratio, the temperature and contact time. Grim (1968) contended that, under certain conditions, weak acids, are also capable of dissolving clays. Hurst (1970) reported use of acetic and/or formic acids to increase permeability of geologic formations.

Malcolm el al., (1975) reported evidence of dissolution of clay minerals by organic permeants. Increasing amounts of Fe^{+3} and Si^{+3} were removed from clay minerals during the passage of an organic permeant containing organic acids, methanol and formaldehyde. They identified organic acid sorption on mineral surfaces as a probable mechanism contributing to clay mineral dissolution.

The slow dissolution of crystalline clay minerals by organic acids, observed by Malcolm et al., (1975), could pose some concern in landfill situations. Chian and DeWalle (1976) and Chian (1976), showed that volatile acid production was high in the first stages of anaerobic decomposition, during the early years of landfill life. High organic acids causing low pH, are capable of dissolving the aluminosilicate clay minerals and destroying the stability of the soil fabric.

3.2 VOLUME CHANGE IN CLAY SOILS

In practice, volume change is seen as:

- compressibility of soil, under applied load,
- swelling, and
- shrinking.

Compressibility results in volume change and volume change in a saturated soil mass can only occur as a result of (Yong et al., 1975):

- pore water extrusion from the macropores (ie. primary consolidation),
- to a smaller extent, compressibility of soil units resulting in water extrusion from the micropores (eg. peds, crumbs & aggregate groups),
- to a very minor extent, the compressibility of individual particles without benefit of water movement.

The extent of contributions from these mechanisms may vary with the soil type and the degree of saturation of the soil. The contributions can be discussed using two models.

Volume changes in a compacted clay liner could occur because of changes in effective stress (Sections 3.2.2) brought about either by changes in external loading applied to the soil or because of changes in environmental conditions (temperature, water table or interaction with leachate) during and after construction.

Compressibility or consolidation and swelling of soil materials are important considerations. Shrinkage can be considered a special case of consolidation, wherein the consolidation pressure is derived from capillary menisci and the surface tension of the water (Mitchell, 1976).

Clay soils undergo volume changes when the water content is changed. When clay soils are dried, shrinkage occurs. Subsequent rewettings cause swelling.

3.2.1 Mechanisms of Volume Change

From the experimental results presented by various research workers (Salas and Serratosa, 1953; Waidelich, 1958; Olson and Mesri, 1970; Sridharan and Rao, 1973) it is evident that the volume change behavior of clay soils is controlled by two mechanisms. In mechanism one, termed mechanical, the volume change is controlled by the shearing resistance at the interparticle level. On the other hand, the physico-chemical mechanism (second mechanism) is controlled primarily by the long range diffuse double layer repulsive forces (Sridharen and Rao, 1973).

3.2.1.1 Mechanical Model

If the load on a laterally confined specimen of soil is increased, the volume of the specimen decreases. This is the direct result of deformations involving physical interaction such as mechanical bending, sliding, rolling and rupture of the particles coupled with shearing displacement at the contacts (Olson and Mesri, 1970). Therefore, the compressibility is controlled by the physical properties of the soil particles, such as, geometric arrangement of particles, particle shape, strength, flexibility and surface friction.

Swelling is assumed to occur because of elastic rebound of deformed particles.

The interaction between adjacent particles are thought to be physical, the separation between particles being less than 10 Angstrom units.

Therefore, the mechanical model is more suited to predict volume change behavior of sands, micaceous silts and soils compacted at high pressures and very low void ratios. Generally, the coarser the gradation, the more important are the physical interactions (Mitchell, 1976).

3.2.1.2 Physico-Chemical Model

Many natural soils are mixtures of clay particles and coarse-grained soil constituents. Therefore, part of their volume change is determined by the mechanical properties of the soil particles with the balance determined by the physico-chemical interaction between the clay particles and the soil solution (Bolt, 1956). Long range electrical forces are needed to explain the various physical and chemical properties of clay suspension. These large range forces are comprised of double layer repulsive forces and van der Waals attractive forces (Olson and Mesri, 1970). Repulsive forces tend to separate the clay particles and the attractive forces tend to bring the clay particles closer together.

Physico-chemical interaction controls the volume change behavior of soils that contain clay minerals capable of developing double layers. Olson and Mesri (1970), contended that compressibility of pure clay mineral kaolinite is controlled by mechanical interactions, whereas compressibility of smectite is physico-chemically controlled. Illite occupies an intermediate position with both mechanical and physico-chemical effects evident during volume change.

3.2.2 Effective Stress Concept in Clay Soil-Water Systems

The concept of effective stress, as formulated by Terzaghi, has made possible the application of rational principles to many soil engineering problems, including volume change behavior (Chattopadhyay, 1972).

Terzaghi realized that the volume change characteristics of soils depends not on the total stress applied to the soil, but rather on the difference between the total stress and pore water pressure created in the pore fluid.

The effective stress for a fully saturated soil, can be stated as,

$$\sigma' = \sigma - u_w \qquad 3.4$$

where,

σ' = effective stress,

σ = applied total stress,

u_w = pore water pressure.

Equation 3.4 assumes that no other forces except the externally applied pressure σ and the hydrostatic pressure u_w exist within the particulate system. It also assumes that the area through which the "intergranular stress" or the effective stress acts is negligible (Sridharan and Rao, 1973).

However, it is well established that both attractive and repulsive forces exist between clay particles. The effective stress σ', defined by equation 3.4, has been found to be inadequate in explaining the volume change characteristics of soils containing active clay minerals because it excludes the forces arising from the peculiar nature of clay minerals and the physico-chemical environment (Balasubramonion, 1972).

3.2.3 The Modified Effective Stress Equation

Because of the strong influence of electrical attractive and repulsive forces at particle level in clays on their engineering behavior, the effective stress concept has been modified to take into account these forces (Lambe, 1960, Mitchell, 1976).

Based on a mechanistic picture ot forces between adjacent particles in a fully saturated clay-water system (Figure 3.1), Mitchell (1976) expressed the modified effective stress concept as,

$$\sigma^* = \sigma' - (R-A) \qquad 3.5$$

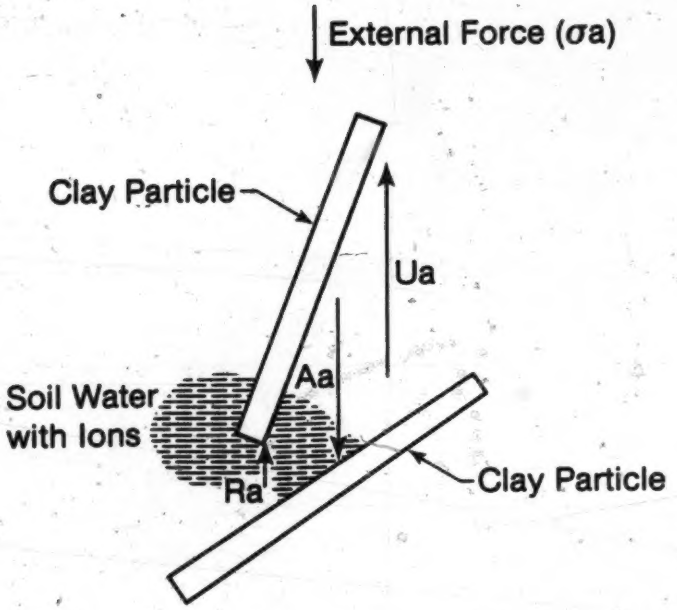
External Force (σa)
Clay Particle
Ua
Aa
Soil Water with Ions
Ra
Clay Particle

Fig 3.1 FORCES ACTING WITHIN A CLAY SOIL

where,

σ^* = true effective stress,

σ' = conventionally defined effective stress (given by equation 3.4),

R = inter-particle repulsive stress,

A = inter-particle attractive stress.

Chattopadhyay (1972), and Balasubramonion (1972) defined (R-A) as the net particle stress caused by the physico-chemical environment, whereas, Sridharan and Rao (1979) termed (R-A) as the intrinsic effective stress. Mitchell (1976) included the attractive force term, A, in an expression defining an intergranular pressure, which is dependent on the long range inter particle attractions ($\bar{A}$) as well as on the applied stress and the pore water pressure between particles.

Equation 3.5 can be rewritten as,

$$\sigma^* - \sigma' = -(R-A) \qquad 3.6$$

When A and R both are small, as would be the case with granular soils, silts and clays of low plasticity, or in cases where A ≅ R true effective stress, σ^*, will be equal to σ', the conventionally defined effective stress or "intergranular stress" (Mitchell, 1976).

Mitchell (1976) stated that (R-A) stress is only important in understanding the compression behavior of well dispersed sodium montmorillonite systems. However, it is evident that, in systems where the physico-chemical model contributes to the volume change (as discussed earlier), a fair understanding of soil behavior, at least qualitative, is forthcoming, when (R-A) stress is also included in the effective stress relationship.

Balasubramonion (1972) examined Equation 3.5 carefully, and noted the following:

- for active clays, σ^* controls the volume change rather than σ'
- at static equilibrium under a total stress, $u_w = 0$, ie. $\sigma^* = (R-A)$,
- for active clays, at very high salt concentration, $R \cong A$, ie. $(R-A) = 0$.

Sridharan and Rao (1973) presented a generalized equation relating the total external stress to internal stresses.

For partly saturated soils,

$$\sigma^* = \sigma - u_w - u_a - R + A \qquad 3.7$$

where,

u_a = effective pore air pressure,
= 0, for fully saturated soil.

3.3 SOIL-WATER ELECTROLYTE SYSTEM

3.3.1 Soil Structure

Soil constituents exist in solid, liquid and gas states. The solid phase is represented by mineral and some organic particles, with size variation from submicroscopic to visually discrete. In general, the liquid phase is a solution of various organic and inorganic ions in water. The gaseous phase approaches the composition of air but with a high content of water vapour. The variability of these three phases, together with corresponding variation in specific interaction of them, provides for the observed characteristics and behavior (Yong and Warkentin, 1975).

The soil particles exist in a certain arrangement and the pore spaces are partly filled with dilute solutions and partly with gases. In saturated soils, these pore spaces or voids are filled entirely with the pore fluids.

Water in the clay soil system plays an important role in determining the behaviour, especially the volume change behaviour of clay soils. Several categories of water in soils are known. Lattice water, interlayer and pore water contribute largely to the physico-chemical forces within the clay-water electrolyte system. The geometric arrangement of particles, particle groups and pore spaces of a soil, is referred to as the soil fabric (see Appendix C for types of fabric observed in natural soils). Two soils can have the same fabric but still exhibit different properties if the forces between particles and particle groups, which govern the stability of the fabric, are not the same (Lambe and Whitman, 1969). The soil fabric and its stability is collectively termed the "soil structure". Soil structure is that property which provides for soil integrity.

The properties of clay soils are determined by the type and amount of clay minerals present and are greatly dependent on the soil structure (the fabric of the clay soil and the interparticle forces). The interparticle forces are greatly influenced by the exchangeable cations on the clay particles and the type and concentration of cations in the pore solution (Ravina, 1972). Therefore, MSW leachates should be capable of changing the interparticle forces and consequently the properties of the clay soils used in constructing liners.

3.3.1.1 Interparticle Forces in Clay-Water Systems

Equation 3.2 has been derived with the reasoning that the repulsive forces tend to separate the clay particles in the same way as positive pore water pressure does, and the attractive forces try to bring them together in the same way as the external total stress does.

Many complex factors are responsible for these net attractive and repulsive forces (Sridharan and Venkatappa Rao, 1979).

Mitchell (1976) reviewed the nature of electrical forces acting between clay particles. Several mechanisms contribute to these electrical forces, and their quantitative evaluation is an unprecedented task.

Of the several interactions contributing to the (R-A) stress, long range particle interactions associated with electrical double layers (repulsive force) and van der Waals attractive forces, are of primary importance, and will be discussed in detail in the following sections.

The long range interactions control the clay particle behaviour in soil suspensions, exclusively. However, in denser soil masses, other forces of interaction become important as well. These short range repulsive and attractive forces will be discussed in section 3.2.3.4.

The quantitative evaluation of (R-A) stress and hence the effective stress σ^* requires the estimation of the following:

- long range repulsive force, manifested by double layer interaction - R_1
- long range van der Waals' attractive force - A_1
- Short range repulsive force (include hydration and born repulsion) - R_2
- short range attractive force (include electrostatic attractions, primary valence and hydrogen bonds) - A_2

In the following sections, an attempt is made to investigate these forces on the following lines:

- quantitative determination (reliability and applicability to this study)
- modification (capacity of leachate to modify these

forces).

However, it is known that long range repulsive forces dominate the interparticle force component. Also, leachate with high electrolyte content is only capable of altering the long range forces. Therefore, consideration of leachate effects should focus on long range repulsion and try to identify the role that other physico chemical forces are playing.

Hence, the following will focus on understanding the nature of the long range repulsive force, R_1, its determination and factors modifying this force.

Repulsive forces between particles are primarily attributed to an interaction between double layers (Lambe,1954). The repulsive force is due mainly to the presence of a layer of ions held closely to each particle by electrostatic attraction, yet prevented by thermal agitation from sticking to it. When two particles approach each other, these layers must be pushed out of the way, and this requires energy.

A particle, with its double layer, is electrically neutral, so that it exerts no net coulombic force upon another. This is the situation at sufficiently large distances between particles. As the particles approach, however, the double layers interpenetrate and interact. When the two surfaces finally touch, there can be no more diffuse layer in between. It is the work required to distort and compress the diffuse double layer which causes most of the repulsion. Therefore, an understanding of the nature of the long range repulsive force requires some knowledge of the electric double layers surrounding clay particles and their interaction with each other.

3.3.1.2 The Electric Double Layer

Although electric neutrality in clay water systems dictates that the sum of exchangeable cation charge equals the negative charge of the soil at a given pH, it is difficult to distinguish between exchangeable and soluble cations (Bohn et al., 1979).

If the double layer contained only those cations necessary to neutralize the colloid charge, the anion concentration would drop to zero at the transition between bulk solution and double layer. However, thermal agitation continually drives anions toward the colloid surface. Therefore, the total negative charge includes that from the soluble anions, plus that of the clay particle. Cations within the double layer must neutralize both sources of negative charge. Part of the cations are exchangeable, because they neutralize the charge of the colloid. The remainder are soluble, since they neutralize the charge of soluble anions.

If the layer of cations balancing the negative charge of the colloid were located immediately adjacent to the particle surface, the distribution would be called a Helmholtz double layer (see Figure 3.2). However, the presence of thermal agitation would generally prevent the

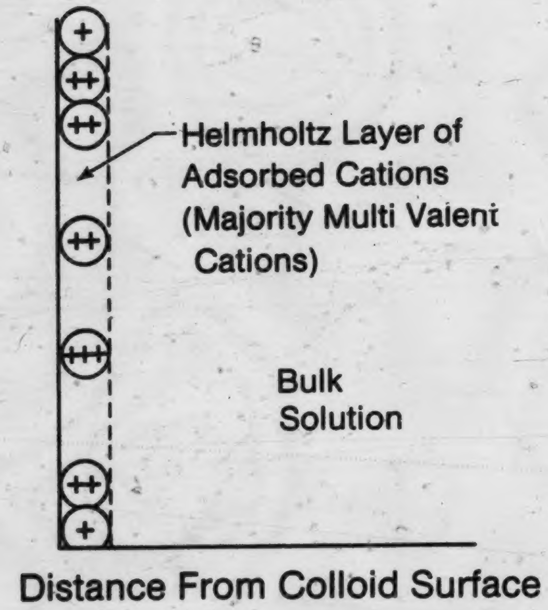
+
++
++
Helmholtz Layer of Adsorbed Cations (Majority Multi Valent Cations)
++
+++
Bulk Solution
++
+
Distance From Colloid Surface

Fig 3.2 DISTRIBUTION OF CATIONS ACCORDING TO THE HELMHOLTZ MODEL

formation of such a compact double layer (Mysels, 1967).

The actual distribution of cations in the liquid phase grades continuously from a high concentration near the particle surface to a lower concentration in the bulk solution. Such a distribution (shown in Figure 3.3) is referred to as the 'diffuse' double layer, or Gouy or Gouy-Chapman double layer, in honour of the two men who first studied it in detail.

The negatively charged particle surface affects both the cation and anions, so that the double layer is formed by both an increase in the concentration of cations and a decrease in the concentration of anions near the particle surface (Mysels, 1967). The electrical field intensity and the potential within such a diffuse double layer decreases gradually from the charged surface to infinity, at first rapidly, then more and more slowly.

As noted earlier, it is the thermal agitation (or the Brownian motion), overcoming, in part, the electrostatic attractions, which causes the diffuseness of the double layer. However, if the electrical forces are too strong, or if they are reinforced by van der Waals forces, thermal agitation would not be able to overcome them, and some of the cations would cluster around the particle forming a compact layer. This compact part of the double layer is called the "Stern" layer.

The Stern model is a modification of the diffuse double layer concept, in which part of the cations are regarded as

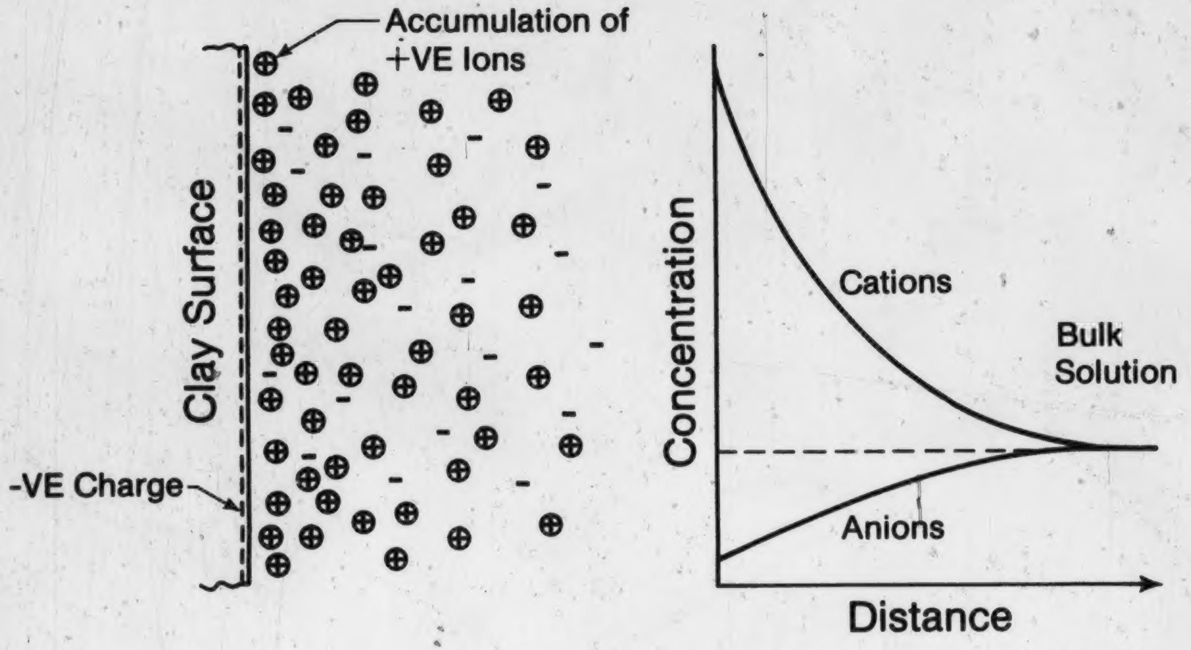
Accumulation of
+VE Ions
Clay Surface
-VE Charge
Concentration
Cations
Bulk
Solution
Anions
Distance

Fig 3.3 DISTRIBUTION OF IONS ADJACENT TO A CLAY SURFACE - Diffuse Double Layer Concept

strongly adsorbed to specific colloid exchange sites. The remaining ions form a diffuse layer of reduced charge density (Bohn et al., 1979).

The structure of the double layer can be represented by Figure 3.4. The figure also illustrates three layers whose potentials are of particular interest.

They are:

ψ_0 - surface potential (measures the total potential of the double layer),

ψ_d - potential at the boundary between the Stern and the diffuse part of the double layer,

ζ - zeta potential, at the "plane of shear" (which essentially separates the water of hydration from free water),

3.3.1.3 Qualitative and Quantitative Aspects of the Diffuse Double Layer

The diffuse double layer has no sharply defined end point but slowly becomes negligible. It is nevertheless very convenient for approximate considerations to assign to it a thickness which is designated by δ. This thickness is generally taken as the distance over which the potential drops to a certain fraction of ψ_d, specifically to $1/e = 0.37$ of ψ_d.

The thickness of the double layer is generally taken as corresponding to the rate of decay of the low potential and of the curve, and is computed from the Debye quantity k_d, as follows:

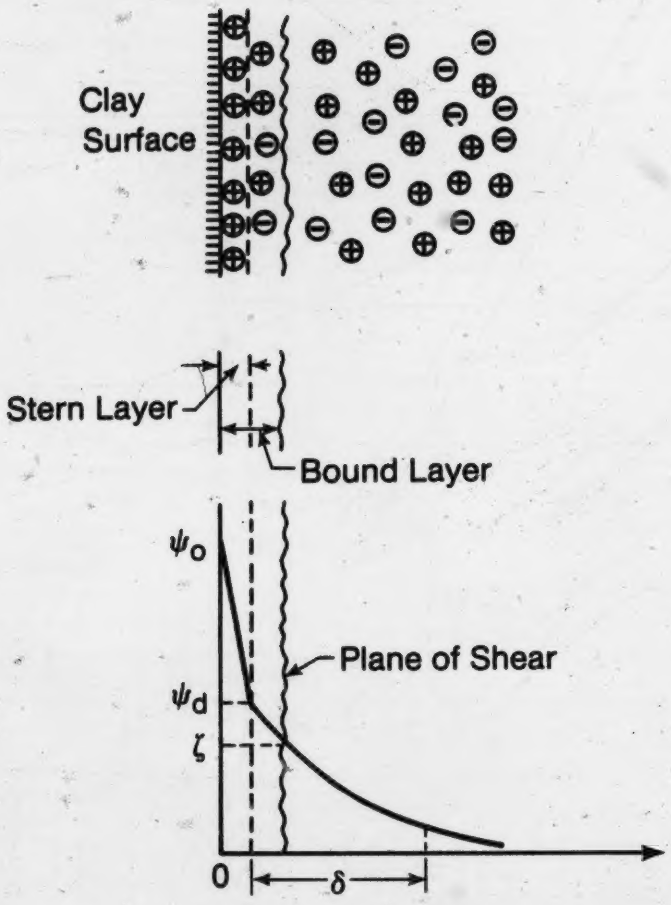
Clay
Surface
Stern Layer
Bound Layer
ψ_o
ψ_d
ζ
Plane of Shear
0
δ

Fig 3.4 THE STRUCTURE OF THE DOUBLE LAYER AND THE CORRESPONDING POTENTIALS

$$k_d = 1/\delta = \{ \frac{8 \pi n}{\epsilon k T} \}^{1/2} Z e \qquad 3.8$$

where,

k = Boltzmann constant,

δ = double layer thickness,

n = concentration of ions,

ε = dielectric constant,

T = absolute temperature,

Z = valence,

e = unit electronic charge.

This thickness changes with the concentration and valence of the ions present in the solution. Double layer thickness decreases as the square root of the bulk solution salt concentration and directly with increasing valence of the exchangeable cation (Equation 3.8).

The effect of increased salt concentration arises from decreased cation diffusion away from the particle surface and increased anion diffusion towards the particle. The effect of cation valence arises from the stronger attraction of higher charged cations by the colloid surface.

Diffuse Double Layer Theory

The exact quantitative treatment of the double layer is complicated (Verway and Overbeek, 1946). Verway and Overbeek (1946) first, applied double layer consideration to develop a basis for the modern colloid stability theory. Later Lambe (1953), in his theory of soil structure, offered a rational explanation of clay properties based on diffuse double layer theory.

In developing the flat plate diffuse double layer theory, Gouy and Chapman made some basic simplifying assumptions (Mitchell, 1960). They include:

- exchangeable cations exist as point charges,
- colloid surfaces are planar and infinite in extent,
- surface charge is distributed uniformly over the entire surface.

Although it is obvious that these assumptions inaccurately describe the actual clay water systems, the diffuse double layer theory seems to work surprisingly well in certain clay water systems. Bolt (1956) examined the validity of these assumptions in detail and suggested that many of the errors inherent in these assumptions tend to cancel with each other, especially at low electrolyte concentrations. Collis-George and Bozeman (1970) extended the application of double layer equations to heterovalent ionic systems, and in the process, identified restrictions of its application to swelling clay systems under a confining pressure.

Equations to describe the theoretical distribution of exchangeable cations, electric potential around a single clay particle together with theoretical distribution of cations between two particles, are given in Appendix D.

Validity of Double Layer Theory and Additional Factors Influencing Behavior

One of the basic assumptions is the consideration of an infinite plate. When we consider a charged sphere, as is the case in most natural clay water systems, the situation is somewhat more complicated. The difference will be quantitative rather than qualitative. The ratio of radius to the thickness of the double layer ($\frac{a}{\delta} = ka$), gives a measure of the curvature within the double layer (Figure 3.5).

If the thickness of the double layer around a sphere is small compared to its radius (a/δ large), its structure will differ little from that of a completely-plane, double layer. This is the case for colloidal systems of larger particles. The infinite plane is not only the simplest model but has a considerable range of applications (Mysels, 1967). It is only when the particle becomes quite small, or the solution very dilute, that the curvature effects become important.

The influence of dielectric constant, coulombic interaction between the ions, and short range repulsion between the surface and ions, were studied in detail by Bolt (1955). The results from this corrected theory are almost the same as those from the simple theory for double layer interaction (Mitchell, 1976).

In the theory of the diffuse double layer, the ions are considered as point charges without dimensions (Kruyt, 1952). As long as the ions are far from the particle surfaces and their concentration is low, their size and

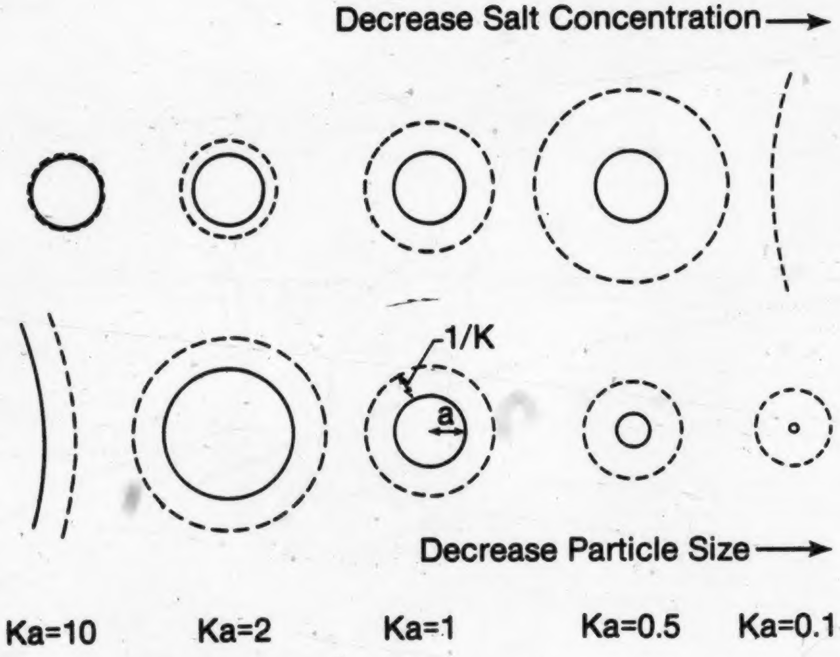
Decrease Salt Concentration
1/K
a
Decrease Particle Size
Ka=10
Ka=2
Ka=1
Ka=0.5
Ka=0.1

Fig 3.5 THE MEANING AND VARIATION OF ka

(adapted from Mysels, 1967)

shape have little influence on the calculations.

Stern improved the Gouy-Chapman theory by assuming that some ions are tightly held immediately next to the colloid surface, in a layer of specifically adsorbed or Stern-layer cations. The double layer is diffused, beyond this layer. This theory is summarized by van Olphen (1963) for both single and interacting flat double layers. The Stern correction reduces the very high concentration predicted near the surface by Gouy's theory and partially explains the differences observed between the adsorption characteristics of monovalent cations (affinity of clay ions is generally $K^+ > Na^+ > Li^+$). Table 3.3 provided the hydrated radii of some of the cations found in soils. Most of the Ca^{+2} ions are held in the Stern layer, while Na^+ ions occur mostly in the diffuse layer (Yong and Warkentin, 1975).

A satisfactory approximation of the Stern model can be made by assuming that the specifically adsorbed ions quantitatively reduce the surface charge density of the colloid. Relatively few of the strongly hydrated Li^+ ions are strongly adsorbed in the Stern layer, compared to the weakly hydrated K^+ ions. Experimental evidence shows that about 20-50% of Na^+ ions are specifically adsorbed, ie., do not dissociate from the particle and participate in mobility phenomenon along the surfaces (Shainberg and Kemper, 1966).

3.3.1.4 Two Particle Interactions

The ion distribution for interacting clay particles, rather than the ion distribution around a single particle, is of interest in the interpretation of soil behavior.

The potential and charge distributions for the case of interacting double layers from parallel flat plates, are shown in Figure 3.6. The distribution of cations, anions and electrical potential in Figure 3.6 are assumed to be symmetrical between two parallel colloid particles.

It is possible to calculate the distribution of electric potential within the double layer for any combination of colloidal charge, electrolyte concentration, counter-ion valence and interparticle distance.

The Boltzmann equation can then be used to calculate cation and anion distributions. From such distributions, colloid swelling can be inferred.

Kemper and Quirk (1970), in an attempt to simplify diffuse double layer computations, presented a set of curves to determine approximate electric potentials as a function of distance from the colloid surface. The available equation, to calculate the electric potential and the ion concentration in the interacting diffuse double layers, is reviewed and presented in Appendix D2. Equation D.16 in Appendix D2 is the most widely used equation:

$$Z \sqrt{\beta C_0}\ (x_0+d) = 2 \sqrt{C_0/C_d} \int \frac{d\phi}{\sqrt{\{ 1 - (C_0/C_d)^2 \sin^2\phi \}}} \qquad 3.9$$

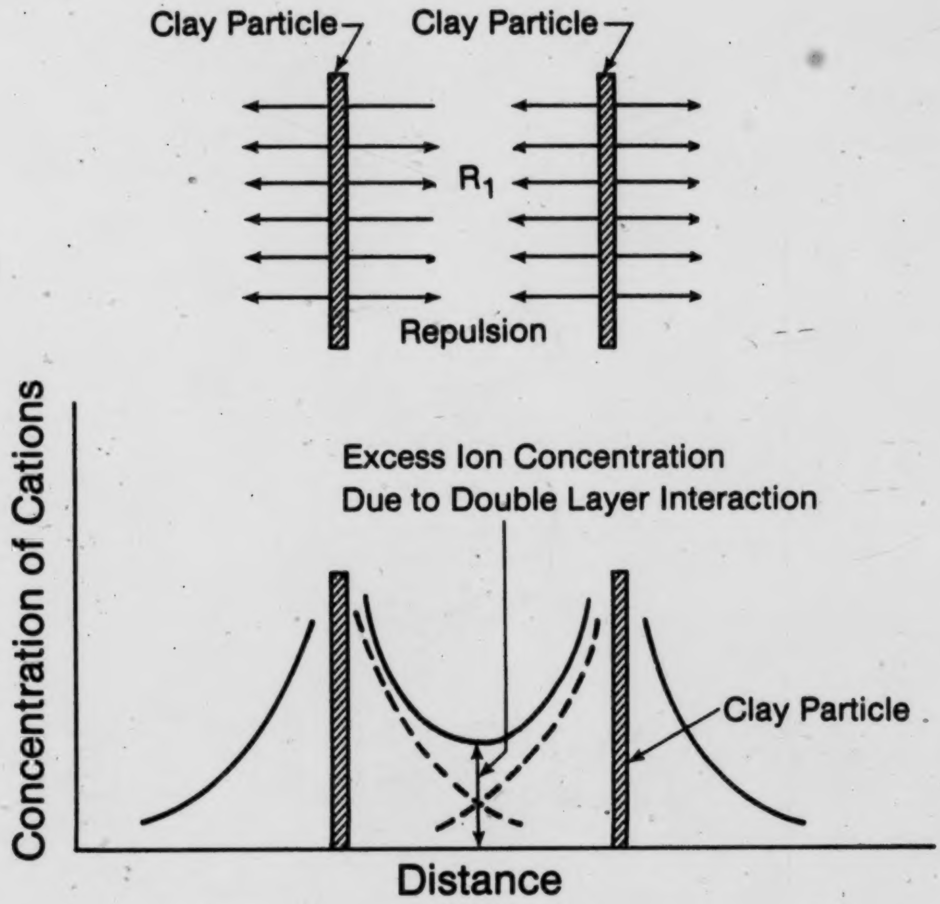
Clay Particle
Clay Particle
R1
Repulsion
Concentration of Cations
Excess Ion Concentration
Due to Double Layer Interaction
Clay Particle
Distance

Fig 3.6 SCHEMATIC REPRESENTATION OF REPULSION BETWEEN CLAY PARTICLES (modified from Yong & Warkentin, 1975)

where,

Z = valence,

β = a constant, depends upon the temperature,

C_0 = cation concentration in the bulk liquid pressed out from the system,

ϕ = variable related to the central concentration,

x_0 = distance from surface,

d = half distance between parallel clay platelets,

C_d = cation concentration in the central plane,

However, a much simpler equation is obtained by neglecting the anion effects in the derivation of the above equation (see Appendix D2). The cation concentration at the plane midway between clay particles, C_d, is given by:

$$C_d = \frac{\pi^2}{\beta Z^2 (d + x_0)^2} \qquad 3.10$$

Equation 3.10 is valid at small distances between particles, since, at low water contents, anions are effectively excluded from the space between particles (Yong and Warkentin, 1975). At very low electrolyte concentrations (C_d much greater than C_0), Equation 3.9 reduces to 3.10.

Table 3.5 compares values of Y_c (Y_c, the reduced electrical potential in the central plane between parallel plates, is related to C_d by $C_d = C_0 \exp(Y_c)$) calculated from the complete solution of the elliptic integral, from the Langmuir approximation and van Olphen method of summing up

TABLE 3.5 VALUES OF Y_c

(Adapted from, Yong & Warkentin, 1975)

Kx	x at C_0 = 0.001 M (A)	Representative water content (%)	Y_c from elliptic integral	Y_c from langmuir equation
4	400	---	0.15	--
3	300	---	0.40	0.10
2	200	150	0.98	1.02
1.5	150	110	1.50	1.48
1.0	100	75	2.30	2.28
0.75	75	56	2.85	2.86
0.50	50	38	3.60	3.68

Calculated for a surface area of 75 m^2/g and parallel arrangement of particles.

the potentials for single particles. At higher Y_c values (greater than 1) Langmuir approximation (Equation 3.10) seems to give comparable results, at least at low C_0 values.

3.3.1.5 Determination of R_1

Interparticle forces of repulsion arise because of the interaction between interpenetrating diffuse ion layers from adjacent particles, from adsorbed water layers and from interaction between like charges.

The initial repulsive force is associated with the hydration of the exchangeable ions. As an ion, situated between two crystals or particles, hydrates (each water molecule requires about 3 Angstroms), a repulsive pressure is created which tends to push the plates apart (Figure 3.6). If there is no opposing plate, because of thermal motion, these ions tend to spread out. The extent of this double layer would be greater than 30 Angstroms. However, the presence of another plate would modify the double layer and the concentration distance function of each of the double layers (Quirk, 1964).

As a result, the midplane concentration would be greater than the external concentrations. This is the basis for the osmotic repulsive pressure which tends to push the particles apart.

This osmotic repulsive stress is found as the difference between the osmotic pressure in the central plane and the osmotic pressure in the bulk solution (Norrish, 1954).

The osmotic force arising from ions in the diffuse double layer, which causes repulsion between charged colloidal surfaces, has first been considered by Langmuir (1938). For a clay mass in equilibrium with a symmetric (1:1) electrolyte, this repulsive stress is given by the van't Hoff Osmotic Pressure equation (Arnold, 1978):

$$R_1 = 2n_0kT\ (\cosh \overline{Y_c} - 1) \qquad 3.11$$

where, T = absolute temperature,

k = Boltzman constant,

n_0 = Concentration in the external solution (in molecules per unit volume).

Since, $R = kN$ and $n_0 = NC_0$ (R and N are, gas constant and Avagadro's number, respectively), it is possible to rewrite the above equation as (Bolt and Miller, 1955):

$$R_1 = 2RTC_0\ (\cosh Y_c - 1) \qquad 3.12$$

For single cation and anion species of the same valence, this equation can be rewritten in terms of midplane cation and equilibrium solution concentrations C_d and C_0 (Sridharan and Jayadeva, 1982), as the van't Hoff equation:

$$R_1 = R\ T\ C_0\ \left(\frac{C_d}{C_0} + \frac{C_0}{C_d} - 2 \right) \qquad 3.13$$

The midplane cation concentration C_d, for known d values, can be calculated from either Equation 3.9 (when Y_c is less than 1) or 3.10.

Half Distance Between Particles, d

For a saturated clay, assuming a homogeneous parallel plate orientation for the clay particles, half the particle spacing is given by:

$$d = \frac{e}{G_s \gamma_w S} \qquad 3.14$$

where,

e = void ratio,

$e = G_s W$, for a saturated soil,

G_s = specific gravity of soil solids,

For S, specific surface of soil solids, measured in square centimeters per gram, $\gamma_w = 1.0\ \text{g/cm}^3$.

Hence, a useful form of the above equation is,

$$\text{d in Angstroms} = \frac{\text{w (\%)} \times 100}{\text{S (m}^2\text{/g)}} \qquad 3.15$$

In deriving Equation 3.14, Bailey (1965) assumed uniform, parallel orientation of infinitely long clay particles, thereby effectively neglecting the edge surface area. These assumptions are not met in natural systems, but may adequately predict interparticle spacing in high swelling Na montmorillonite at low salt concentrations.

Determination of an effective half spacing, d, for natural systems, containing silt and sand size particles, domains or tactoids, have been attempted by Balasubramonian (1972), Yong and Warkentin (1975). By taking into account the "dead volume", random arrangement of particles, and existence of particle clusters as tactoids, a better estimate of effective half spacing d can be obtained.

Application in Natural Systems

The preceding relationships, for the determination of C_d and repulsive stress R_1, were developed for soils containing a single electrolyte. Approximate equations for mixed cation systems, that cover most of the natural systems, are available in soil science literature (Collis-George and Bozeman, 1970).

Elliott (1977), developed relationships for the half distance, free energy and repulsion for interacting double layers in symmetrical mixed ion systems. However, most of the natural systems, containing sodium, potassium, calcium, magnesium chloride, and sulphates most closely fit the case of non symmetrical mixed ion systems. But, the error in considering these situations as symmetrical mixed ion systems are small (less than 3 percent), and is of less consequence than errors due to other assumptions (Elliott, 1977).

Repulsive stress between interacting particles, as calculated using this procedure, depends upon the validity of the van't Hoff relationship. However, the expected deviation from this law may be partly offset by the effect of non-ideal behavior of the ions in the double layer and the relative magnitude of the error will probably not exceed 10 percent (Bolt, 1955).

3.3.2 Attractive and Short Range Repulsive Forces

The colloidal particles in the majority of soils in their natural state are flocculated, and clustered together in aggregates. Clays are normally held together in tactoids or domains, and these domains are bonded to form aggregates.

Historically, there was much interest in repulsive forces between soil particles, because of their contribution to soil swelling and dispersion. However, it is the balance between van der Waals attractive forces, and several repulsive forces including electric double layer repulsion and other short range repulsive forces, that determine whether the soil would disperse or flocculate (Arnold, 1978).

3.3.2.1 Attractive Forces

The phenomenon of flocculation demonstrates the existence of interparticle attractive forces. In order to compete with the double-layer repulsion, which is still operative under flocculating conditions, the attraction must be of comparable range and magnitude. The general van der Waals attractive forces appear to satisfy this requirement (van Olphen, 1963).

The van der Waals attractive forces between atoms (which are the result of dipole interactions), are small and decay rapidly with distance. This attractive force is inversely proportional to the seventh power of the distance for two atoms. For a collection of molecules, as for particles containing a large number of atoms, this attractive force is approximately additive so that the attractive force between particles is equal to the sum of the attractive forces between all the molecular pairs (Arnold, 1978).

Therefore, the attractive force between two spherical particles is inversely proportional to the third power of distance. The attractive energy, V_A, for such particles is thus inversely proportional to the square of the distance of separation (Gast, 1977), and is given by:

$$V_A = \frac{-A}{48\pi d^2} \qquad 3.16$$

where, A is known as the "Hamaker" constant, and d is the half distance between parallel particles.

Taking the thickness of the clay platelet into consideration, the above equation can be rewritten for attractive force (Sridharan and Rao, 1979) F_A as:

$$F_A = \frac{A}{48\pi}\left[\frac{1}{d^3} + \frac{1}{(d + \delta)^3} - \frac{2}{(d + 0.5\delta)^3}\right] \qquad 3.17$$

in which, δ is the clay particle thickness.

There has been much discussion concerning a suitable value for the "Hamaker" constant A. The value of A depends on the nature, including the polarizability, of the particles and the medium separating them. Sridharan and Rao (1979) showed the A parameter to be inversely proportional to the dielectric constant of the medium.

Values of A ranging from 10^{-11} to $5\text{x}10^{-14}$ ergs are reported in literature (Mitchell, 1976). Sridharan and Jayadeva (1982) computed a value of $6.11\text{x}10^{-13}$ ergs for montmorillonite and kaolinite in water.

Assuming particle thicknesses of 1 nm and 100 nm for montmorillonite and kaolinite respectively, Sridharan and Jayadeva (1982) computed attractive pressures using Equation 3.17. They concluded that (at least for their system), in relation to the repulsive pressure calculated by the double layer theory, the contribution of the van der Waals attractive pressure is negligible and could be ignored. They came to this conclusion by studying the void ratio (e) vs log effective stress (log p) for both montmorillonite and kaolinite clays. In both cases, there was no discernible effect on the e vs log p curve as a result of adding the extra attractive stress term (Figure 3.7).

In both fresh and salt water, interparticle attraction and repulsion operate simultaneously. However, unlike repulsion, which decreases with increasing salt

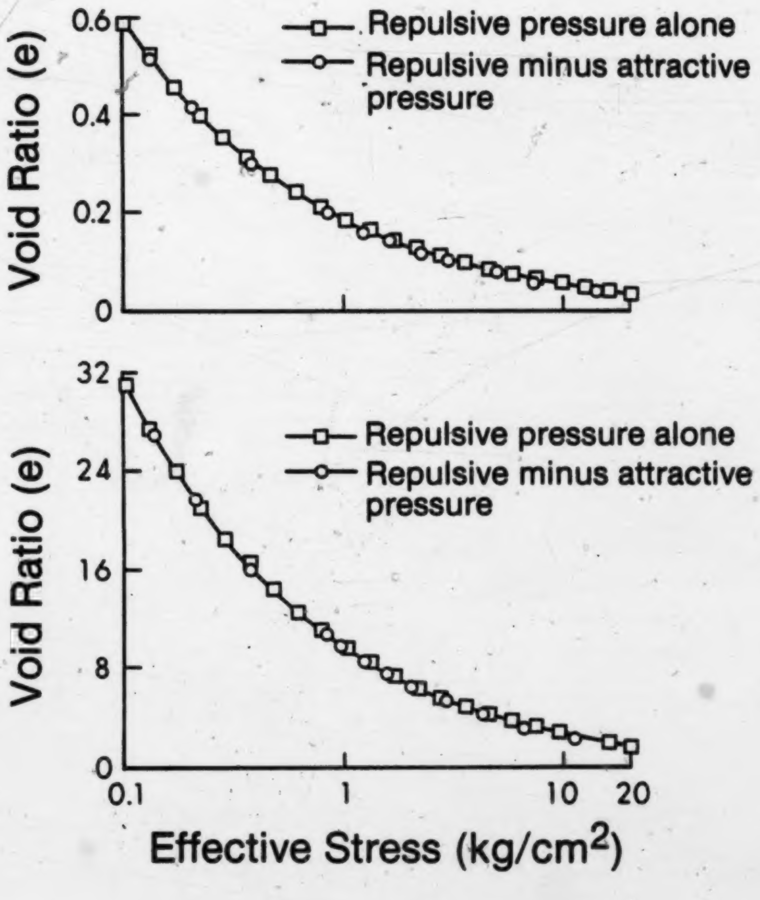

Fig 3.7 EFFECT OF ADDING THE " A STRESS ":

a) kaolinite,

b) montmorillonite,

(adapted from Sridharan and Jayadeva, 1982) concentration, attraction remains practically the same when the electrolyte concentration is varied (Fig. 3.8). In fresh water, the repulsion predominates, but in salt water the repulsion is reduced to the point at which the attraction begins to dominate, and the soil flocculates (van Olphen, 1963).

The counter ions in most soil systems are dominated by Ca^{+2}, Mg^{+2}, H^{+} or Al^{+3}, all of which tend to reduce the electrical double layer repulsive forces to the point they are less than the van der Waals attractive forces (Gast, 1977). The major exception is in the case of soils with high Na^{+} saturation.

3.3.2.2 Short-Range Repulsive Forces

As indicated above, the electric double layer repulsive forces in most natural clay soils are less than the van der Waals attractive forces, resulting in a net particle attraction. In such situations, the clay particles approach each other until the net attractive forces are balanced by the short range repulsive forces (Gast, 1977).

The short range repulsive forces are a combination of Born repulsive forces and specific adsorption forces.

Born repulsive forces are due to resistance on interpenetration of crystal structures, and becomes effective as soon as extruding lattice points or regions come into contact (van Olpen, 1963).

Specific adsorption forces are the result of molecular adsorption by the particle surface. This involves water

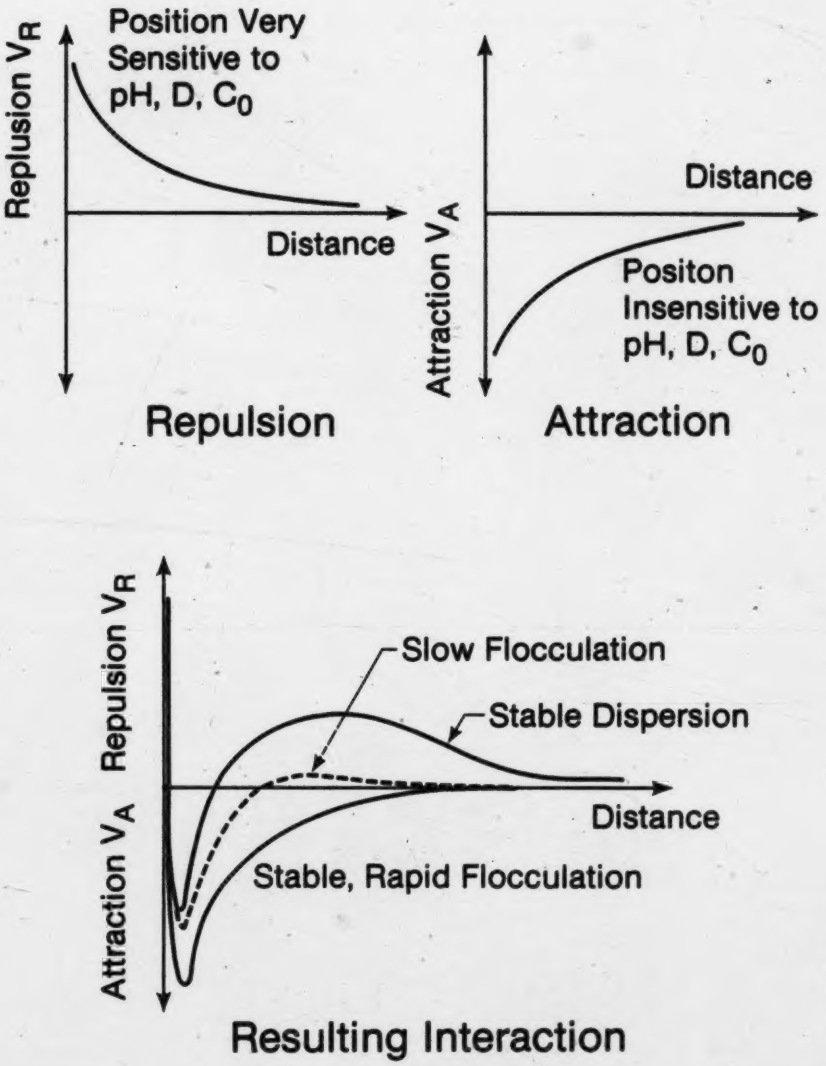
Replusion V_R
Position Very Sensitive to pH, D, C_0
Distance
Repulsion
Attraction V_A
Distance
Positon Insensitive to pH, D, C_0
Attraction
Repulsion V_R
Attraction V_A
Slow Flocculation
Stable Dispersion
Distance
Stable, Rapid Flocculation
Resulting Interaction

Fig 3.8 FORCES OF INTERACTION

(modified from Mitchell, 1976)

molecules either adsorbed on the surface or in the hydration shells of the counter ion. The energy required to remove these adsorbed water molecules, before the distance between the particles can become less than the thickness of the hydration shell, is reflected by a short range repulsion between particles (Gast, 1977).

These hydration repulsions decay rapidly with increasing particle separation, varying inversely as the square of the distance or less (Mitchell, 1976).

3.3.3 Interparticle Forces and Clay Swelling

Apart from mechanical effects, as reflected by elastic rebound during stress relief, swelling generally results from several physico-chemical causes, including: surface hydration, cation hydration and osmotic pressure effects (Mitchell, 1973). Forces of hydration (surface and cation) are normally satisfied after adsorption of the first few water layers. Major contribution to swelling in highly expansive soils is from the physico-chemical interactions and osmotic effects.

A clear understanding of the relative importance of the attractive and repulsive forces can be obtained by studying the swelling behavior of montmorillonite clays. Norrish (1972) considered three stages of swelling of montmorillonite (often taken as a model material to discuss swelling) in an idealized description of swelling.

In the first region, interlayer spacing of montmorillonite increases stepwise to about 20 Angstroms, from a dry spacing of about 10 Angstroms, during initial wetting. This swelling is very much dependent on the cation exchange capacity of the clay mineral and the interlayer cations. The swelling is primarily due to the hydration of cations (Mitchell, 1973), but the clay surface, with its high affinity for water molecules, may also contribute to swelling at this stage (Barshad, 1952).

The electrostatic attraction between negatively charged sheets, and positively charged interlayer cations, provide the attractive force necessary to counter the repulsion due to cation hydration. This interlayer swelling is common in montmorillonite-type clays, but virtually non-existent in 1:1 clays, such as kaolinite.

If the interlayer cations are monovalent and have sufficient hydration energy, swelling proceeds to the next stage (Norrish, 1972).

In the second region, increase in particle spacing is normally explained by the long range diffuse double layer repulsion. Since the increase in layer separation (sometimes up to several hundred Angstroms) is due to the formation of diffuse double layers between individual silicate sheets, this swelling can be controlled by the addition of salt solutions. The interlayer separation, due to diffuse double layers, would be inversely proportional to the square root of the salt concentration (Norrish, 1972). Furthermore, interlayer swelling observed during this stage of expansion is linearly related to the water content of the clay water system (Fink and Nakayama, 1972).

Seed _et al._, (1962) explained swelling in this region based on the "osmotic pressure" concept. The adsorbed cations, once dissociated from the particle, form an ion atmosphere in which the osmotic pressure would be greater than outside. This pressure would cause water to flow from the point of low concentration to the point of high concentration (Bolt, 1956). Grim (1968) termed this ion atmosphere and the osmotically imbibed water surrounding the particle, the "clay micelle" (water and ions within the micelle constitute the diffuse double layer). In this region, swelling is opposed not only by van der Waals attractive forces, but also by frictional forces as brought about by edge to face bonds (Emerson, 1963).

However, non-expandable clay minerals (kaolinite and illite), saturated with monovalent cations, can only form diffuse double layers on the external surfaces of the clay particles (Aylmore and Quirk, 1959), and it seems doubtful if diffuse double layers form with any clays saturated with divalent cations (Aylmore and Quirk, 1962). The interlayer spacing for Ca montmorillonite does not exceed 20 Angstroms. These Ca montmorillonite particles are capable of aggregating themselves into layer units even in salt free water (Shainberg and Kaiserman, 1969).

In a third stage in the absence of frictional forces, the clay plates would separate to very large distances and go into suspension.

3.3.4 Volume Shrinkage and Crack Formation

In practice, shrinkage cracks within clay soil systems are formed in response to the tensions produced within a soil matrix, created by volumetric decrease (Plummer and Gostin, 1981). This volume decrease is induced by loss of water from the soil matrix, which is the direct result of variations in soil compaction (Fuller and Warrick, 1985), temperature (Hamilton, 1966) and salinity (Gray, 1984) in the pore water.

Dessication cracks caused by soil freezing, ice wedging, frost fissuring and drying occur at the soil-air-interface as a result of subaerial exposure. But, synaerisis cracking, caused by salinity changes, can occur subaqueously (Burst, 1965). Plummer and Gostin (1981) conducted an extensive investigation on the most likely causes of shrinkage cracks, in sedimentary deposits. They showed that shrinkage cracks can form, not only on the sediment-air interface by dessication processes, but also at the sediment-water interface, sub-stratally, by synaerisis.

A summary of shrinkage crack origins and characteristics is shown in Table 3.6.

TABLE 3.6

SHRINKAGE CRACKS - ORIGINS AND CHARACTERISTICS
(modified from Plummer and Gostin, 1981)

TYPE OF SHRINKAGE CRACK AND PLAN SHAPE	X - SECTIONAL SHAPE	NUMBER OF GENERATIONS OF CRACKS	ORIGIN AND DEPOSITIONAL ENVIRONMENT
DESSICATION	V or U Shaped	can be multiple	Algal or mud shrinkage by atmospheric drying (exposed)
SYNAERISIS		generally one	Surface or substratal dewatering of submerged muds (submerged)

3.3.4.1 Soil Water

The preceding discussion indicates that clay particles present in a soil-electrolyte system are always hydrated; that is surrounded by layers of water molecules.

Three categories of water in soils are recognized by soil engineers:

- water present in the crystal lattice,
- interlayer water between the basic structural units of montmorillonite,
- pore water, which may be of two types; bulk or free water and adsorbed water (within the "clay micelle").

Grim (1968) defined "adsorbed water" as the water influenced by the clay particle charge or simply water within the "clay micelle" (Figure 3.9). The water and ions within the micelle constitute the diffuse double layer.

Lattice water is strongly attracted to the mineral particles and very high energies are required to remove it. Loss of interlayer and pore water is expected in practice and causes the volume shrinkage and subsequent cracking in clay soils.

3.3.4.2 Mechanisms of Shrinkage and Crack Formation

Several phases of shrinkage, "structural", "normal" and "residual", are reported in literature (De Jong and Warkentin, 1965).

These different phases of shrinkage are clearly visible during the process of thermal dessication of a field soil

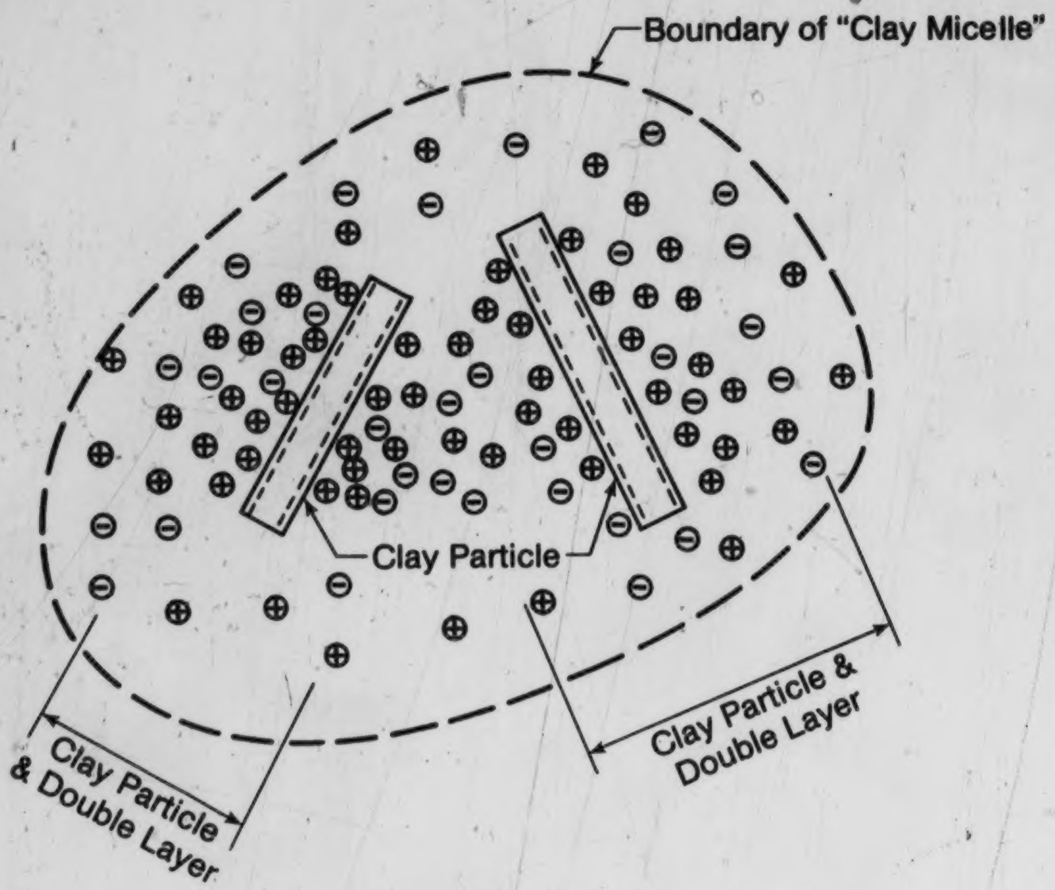
Boundary of "Clay Micelle"
Clay Particle
Clay Particle & Double Layer
Clay Particle & Double Layer

Fig. 3.9 "CLAY MICELLE"

sample.

Structural shrinkage is observed at the beginning of the dessication process. Because of high water content, the spacing of the particles is comparatively large and the tension in the water is small. Because of the presence of large amounts of water, the bond between water and the clay particles in the outer fringes of adsorbed water is small. Therefore, molecules of water from this region are easily removed and dissipated into the surrounding atmosphere. This stage, where only a few large pores are emptied, is observed only in the surface soils with a stable aggregate structure (Sridharan and Rao, 1971).

As dessication, or loss of pore water proceeds, shrinkage enters the normal phase, where menisci begin to form and the surface tension, with progressive loss of pore water due to dessication, increases the magnitude of negative pore water pressure. This will result in developing forces which draw the particles into closer proximity with one another, causing a reduction in total volume.

Dessication of plastic clay continues until the soil mass comes to the shrinkage limit when air starts entering the voids. Particles come into contact with each other during the third phase of shrinkage, called the "residual shrinkage". During residual shrinkage, volume reduction occurs because of further fabric arrangement and bending of particles.

In the normal shrinkage phase, volume decease is equal to the volume of water lost. In both structural and residual shrinkage phases, the decrease in soil volume is much smaller than the volume of water lost.

Berndt and Coughlan (1976) noted that the normal shrinkage accounts for almost 95% of the total volume change in unsupported cores. Furthermore, their observations suggested that the volume change process in the field is primarily "normal" and three dimensional.

Normal Shrinkage and the Effective Stress Concept

Terzaghi (1925) was the first to realize the similarity between the processes of consolidation and shrinkage. He went on to state that the only difference between consolidation and the natural shrinkage is that compression under load can be externally manipulated, whereas natural shrinkage is also governed by internal stresses.

Normal shrinkage can be viewed in the light of the effective stress equation (Equation 3.5), which was derived by equating the total external stresses to the internal stresses in a soil system. For an initially saturated soil mass, at equilibrium with its stress environment ($u_w = 0$), one can rewrite the Equation 3.5 as,

$$\sigma^* = \sigma - (R-A) \qquad 3.18$$

As noted earlier in the discussion of normal shrinkage, during dessication, as evaporation removes water from the surface, a meniscus begins to form in each of the pores at the surface; this results in developing tensile forces in the water.

Therefore, during normal shrinkage, the hydrostatic stress difference required for the outward flow is caused by the surface tension of the capillary water. This surface tension creates the capillary forces, which should be included in Equation 3.16 to build a comprehensive equation capable of explaining the shrinkage phenomenon.

Denoting surface tension forces as σ_s, the following equation, which controls the internal shearing resistance in a shrinkage soil, can now be written (Sridharan and Rao, 1971).

$$\sigma^* = \sigma + \sigma_s - (R-A) \qquad 3.19$$

If the externally applied force is kept constant, normal shrinkage would occur as a result of changes in σ_s or (R-A), or both.

During dessication shrinkage, the volume changes occur as long as the capillary forces are larger than the internal shearing resistance, which is governed by Equation 3.17. As the volume change continues, the internal resistance becomes larger because of increased particle contact (increases A in Equation 3.15), and the capillary forces also increase as a result of further decrease of the curvature of the menisci. Therefore, dessication shrinkage (due to drying) can be explained as being initiated by the capillary forces caused by surface tension of the pore fluid, but the resultant shrinkage is dependent on the resistance offered by the clay water system. This resistance is controlled by the physico-chemical forces, R and A.

3.3.4.3 Synaerisis Shrinkage

Synaerisis is the spontaneous separation of an initially homogeneous colloid system into both a coherent gel and a liquid. This phenomenon basically involves the drawing together of particles under the action of increased attractive forces (Krynine and Judd, 1957). This would invariably result in the formation of cracks.

Several workers (White, 1961; Burst, 1965; Plummer and Gostin, 1981; Yanful, 1982) have observed synaerisis cracks in clays suspended in salt solutions. White (1961) thought that the formation of the cracks was due to the expulsion of water from the clay-water system by internal forces. Komex Consultants (1985) reported Crooks' (1981) observations, where horizontal cracks were observed in bentonite clay compacted by hand in beakers containing $CaCl_2$ solutions of concentration 0.1M.

Turk (1974), studying leakage of concentrated brines from solar ponds, observed subaqueously formed shrinkage cracks. Differential subaqueous shrinkage of fine grained sediments, which caused an extensive fracture system to develop, was attributed to a combination of dessication and shrinkage.

Synaerisis Shrinkage and the Effective Stress Concept

As in the case of dessication shrinkage, synaerisis shrinkage can be explained by the effective stress equation and the concept of normal shrinkage.

As noted earlier, increase in the pore water cation concentration results in the decrease of the long range repulsive stress, R. The degree of reduction depends on the initial repulsive stress and the type and concentration of cations. The reduction in R invariably causes the net repulsive stress term, (R-A) in Equation 3.16 to be diminished and thereby increase the true effective stress σ^*. Equations, proposed in Appendix D and 3.5 can be used to calculate the change in σ^* caused by such an action. The increase in σ^* would cause the pore water to be squeezed out of the soil matrix and be dispersed into the surrounding environment.

The increasing release of physico-chemically bound water, from the clay micelle, to the pore spaces causes more liquid to be drawn out of the clay-electrolyte system. This could go on until the net repulsive stress ((R-A) stress) reaches zero, at which point, the effect of cations on the physico-chemical forces would be minimal. The differential subaqueous shrinkage, that could accompany synaerisis, causes synaerisis cracks.

3.3.4.4 Effect of Different Fabric Conditions on Shrinkage

When the net (R-A) is large, as in the case with a relatively oriented (dispersed) soil structure, lesser shear resistance is offered. However, the sample with the relatively oriented fabric contains a greater number of pores of smaller radius; this gives rise to larger capillary forces during the dessication process. Hence, forces causing volume change increases and the forces responsible for shear resistance decreases. This results in larger shrinkage.

Thus, it is seen that, relatively oriented fabrics are susceptible to more shrinkage during drying than relatively flocculated structures. Furthermore, soils with larger R values are prone to larger volume changes, both as a result of dessication (less resistance offered) and synaerisis (more chance of R reduction).

Summary

When designing a landfill for leachate containment, considerable attention should be given to the clay leachate interactions that are capable of altering clay soil properties. MSW leachates, containing large amounts of cations, can alter the interparticle stresses within a clay soil system and produce synaerisis shrinkage. Since synaerisis shrinkage could result in the formation of new cracks and widening of existing cracks in a compacted clay liner, a clear understanding of the mechanism involved in this mode of soil shrinkage is necessary, if one has to ensure adequate liner integrity, until the pollution threat from the landfill leachate ceases.

4. EXPERIMENTAL PROGRAM

4.1 INTRODUCTION

The experimental program was designed to study the synaerisis shrinkage effect of simulated MSW landfill leachates. First, the effects on specific clay minerals were studied. Later, natural soils obtained from Alberta Environment were evaluated by the procedures developed with the clay minerals.

Factorial designs were used as a means of studying the relative influence of different cations and concentrations in the leachate, exchangeable cations and clay mineral types in the soil samples, and their interaction effects.

A modified consolidation experiment, hereby called the synaerisis shrinkage test, was considered the basic test necessary to study the synaerisis phenomenon. The much simpler "sedimentation test", which is a modification of the "free swell test" proposed by Holtz & Gibbs (1956), was performed concurrently. Long term permeability tests were also performed for a very limited number of test samples, by modifying the apparatus used in the synaerisis shrinkage test. The synaerisis shrinkage test apparatus, which is essentially a consolidometer, is discussed in Section 4.3.1.

4.2 PROPERTIES OF THE SOILS STUDIED

The following soils were tested during the research program:

a) Soil mixtures, prepared using commercially available clay minerals,

b) Locally obtained natural soils, and

c) Modified natural soils (with pure clay minerals).

To investigate the fundamental nature of synaerisis shrinkage, it was necessary to perform experiments on pure clay minerals, rather than on natural soils with mixed mineralogical compositions. Use of pure clay minerals is necessary for any theoretical investigation. Usage of natural soils would make unmanageable the interpretations necessary for establishing a physical basis of the synaerisis shrinkage phenomenon.

Clay minerals are the most important component of clay soils and clay shales (as far as the physico-chemical forces in soils are concerned). They control the engineering properties, permeability, strength and compressibility of soils.

4.2.1 Clay Minerals

The most common clay minerals found in natural soils of Alberta are kaolinite, montmorillonites and illites. However, only kaolinite and montmorillonites were available for this study. Illites could not be obtained commercially, in adequate purity.

From the analysis presented in chapter three, it is obvious that not only the type of clay mineral but also the type of exchangeable cation, controls the interparticle forces, and thereby the synaerisis shrinkage phenomenon in clay soils. Therefore, montmorillonites with two different exchangeable cations, sodium and calcium were obtained and used in the study.

Kaolinites (containing almost 100 percent kaolin) were obtained from Alberta Ceramics. The sodium version of montmorillonite, was obtained as bentonite from Dresser Magcobar (Canada) Ltd. An x-ray diffraction study showed the bentonite supplied by Dresser Magcobar (Canada) Ltd to contain approximately 85% montmorillonite and 15% illites in the less than 2 micron fraction. American Colloid Company supplied the Ca montmorillonite, which was comprised almost entirely of montmorillonites.

4.2.2 Natural Soils

During the research program, 3 natural soils were studied. Two of them were supplied by Alberta Environment and the third sample was obtained from the Geotechnical Division of the Civil Engineering Department. They contained moderate amounts of clay sized particles. The predominant clay mineral was Illite. All three samples seemed to originate from glacial tills of Alberta (see also appendix B2 - clay soils in Alberta).

4.2.3 Classification Tests

4.2.3.1 Clay Minerals

The mineralogical composition of the clay minerals, based on x-ray diffraction analysis, are reported in Table 4.1. The other routine classification tests, including the Atterberg limits and specific gravity of the soil solids, were carried out in accordance with the procedures outlined in the manual on soil sampling and methods of analysis (Mckeague, 1978). The results are presented in Table 4.2.

All three clay minerals contained a very high percentage of fines (silts or clays, according to the unified classification system), more than 95% passing through the #200 sieve. Clay sized particles exceeded 80%, the bentonites showing the highest percentage. The presence of very high percentages of less than 2 microns (or clay sized) particles in these soils, warrant them to be categorized as pure clay minerals.

4.2.3.2 Natural Soils

These soils did not contain very high percentages of fines. About 48% of soil C passed through #200 sieves, whereas for soil A and B, the percentage passing #200 sieve was 41.5 and 43, respectively.

Using the method presented by Mckeague (1978), less than 2 micron or clay sized fractions were collected and sent to the Alberta Research Council to be analyzed for the minerological compositions. These results are presented in

TABLE 4.1

COMPOSITION OF THE CLAY MINERALS

CLAY MINERAL	CLAY MINERAL COMPOSITION (%)			MASSIVE MINERAL COMPOSITION	
	Kaolinite	Montmorillonite	Illite	Quartz	Feldspar
Kaolinite from Alberta Ceramics	100	---	--		
Commercial Bentonite (Wyoming)	---	85	15	Both present in very small amounts	
Ca Montmorillonite from American Colloid Co.	---	100	--	Both present in very small amounts	

TABLE 4.2

PHYSICAL PROPERTIES OF THE CLAY MINERALS

CLAY MINERAL	% CLAY SIZES	SPECIFIC GRAVITY OF THE SOIL SOLIDS (G_s)	INDEX PROPERTIES			
			Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Activity
Kaolinite from Alberta Ceramics	83	2.64	66	42	24	0.29
Commercial Bentonite (Wyoming)	90	2.75	646	80	566	6.29
Ca Montmorillonite from American Colloid Co.	80	2.74	91	48	43	0.54

Table 4.3. Data obtained by performing routine classification tests on the three natural soils are included in Table 4.4.

4.2.4 Exchangeable Cations and Cation Exchange Capacity

Cations present in the exchangeable complex of the clay minerals play an important role in determining the engineering behavior of the clay mineral.

Exchangeable cations in the pure clay minerals and the natural soils were determined by NH_4^+ saturation of the soil and the subsequent displacement of exchangeable cations. The procedure adopted was originally proposed by Chapman (1965) and is outlined below.

Air dried soil, passing #200 sieve (less than 63 micron size) was first treated with H_2O_2 to remove the organic carbon present in the soil. This step was more important in natural soils rather than in pure clay minerals, since natural soils were found to contain more organic matter in the less than 63 micrometer size fraction. Following treatment, the soil was washed and air dried again.

10 g of this air-dry soil was transferred into a 250 ml Erlenmeyer flask and 50 ml of 1N NH_4OAc at pH 7 was added. The flask was shaken thoroughly for a few minutes and then allowed to stand overnight. Before starting the filtration step, the Erlenmeyer flask containing the soil was again shaken thoroughly. The contents were transferred to a 55 mm Buchner funnel in which a moist Whatman #42 filter paper has

TABLE 4.3

COMPOSITION OF THE NATIVE SOILS

	CLAY MINERAL COMPOSITION				OTHER MINERALS
	Illite	Chlorite	Montmorillonite	Kaolinite	
SOIL A, from Alberta Environment	55	5	10	30	Quartz, Feldspar and Dolomite
Soil B, from Alberta Environment	40	5	20	35	Quartz, Feldspar and Dolomite
Soil C, from Civil Eng. Department, University of Alberta	45	5	20	30	Quartz and Feldspar

TABLE 4.4

PHYSICAL PROPERTIES OF THE NATIVE SOILS

CLAY MINERAL	INDEX PROPERTIES		
	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTICITY INDEX %
SOIL A	38.5	27.0	11.5
SOIL B	43.0	26.0	17.0
SOIL C	41.0	26.0	15.0

been seated by gentle suction. The contents were rinsed into the funnel with NH_4OAc. Thereafter, the soil was leached with small portions of NH_4OAc until the collected volume was approximately 225 ml. It was necessary to apply a medium to high suction pressure to force the NH_4OAc through the densely packed soil. The difficulty in leaching was very much evident in the bentonite sample, which required almost 12 hours of leaching. Special care was taken to prevent soil drying and subsequent crack formation during the entire leaching process.

The leachate, once collected, was transferred to a 250 ml volumetric flask and made to volume with NH_4OAc. This extract was analyzed for Na, Ca, Mg and K using the atomic absorption spectrophotometer. The results are expressed as meq/100 g of dry soil and presented in Table 4.5.

In the next step, NH_4-leached soil was used in determining the total cation exchange capacity of the soil.

The funnel containing the NH_4 leached soil was replaced on the suction flask and was washed four times with neutral 1N NH_4Cl and once with 0.25N NH_4Cl. The leachate was discarded. After saturating the soil with NH_4^+ ions, excess electrolyte was washed away with 200 ml of 99% isopropyl alcohol.

The ammonium saturated soil was allowed to stand and drain thoroughly, but before it dried out, 10% acidified NaCl was passed through the soil. Small portions of NaCl were leached at a time, allowing each portion to pass

TABLE 4.5

CHEMICAL PROPERTIES OF THE CLAY MINERALS AND NATIVE SOILS a) EXCHANGEABLE CATIONS

SOIL MATERIAL	EXCHANGEABLE CATIONS (meq/100 g of dry soil)				CATION EXCHANGE CAPACITY (meq/100 g of dry soil)
	Na	Ca	Mg	K	
Kaolinite	8.70	3.12	1.23	0.12	13.17
Bentonite	53.4 + 0.4	2.63	8.94	2.06	67.17 + 0.40
Ca Montmorillonite	0.35	100.0 + 1.0	10.49	1.11	111.94 + 1.0
Soil A	2.72	17.63 + 0.32	3.50	0.24	24.09 + 0.32
Soil B	0.15	19.32 + 0.63	1.97	0.18	21.62 + 0.63
Soil C	3.87	11.75 + 0.25	2.68	0.20	18.51 + 0.25

through the sample before more is added. About 225 ml of extract was collected following this procedure. The NH_4^+ concentration in this extract was determined and used to calculate the cation exchange capacity of the soil. These results are presented in Table 4.5, along with exchangeable cation concentrations.

4.2.5 Soluble Salts

From the several methods available, the most desirable method for determining soluble salts in pure clay minerals seems to be the saturation extract method. However, as evident from the discussion on consolidation cell sample preparation procedures (Section 4.3.2.1), a saturation extract of the test soil could be obtained during initial stages of slurry consolidation. Soluble salts were determined for extracts obtained from the standard procedure (McKeague, 1978) and presented in Table 4.6.

The saturation extract procedure involves the preparation of a saturated paste of the soil and subsequent application of a high vacumm pressure to extract the pore water. This procedure, of extracting pore water from a saturated soil, is more suitable for natural soils containing moderate to low levels of clay minerals. However, in the case of pure clay mineral bentonite, squeezing out all the pore water by the application of vacuum pressure was not possible. Therefore it was decided to use the consolidometer as a device for squeezing out the pore water.

TABLE 4.6

CHEMICAL PROPERTIES OF THE CLAY MINERALS AND NATIVE SOILS b) SOLUBLE SALT DATA

SOIL MATERIAL	CATONIC COMPONENTS OF SOLUBLE SALTS IN THE SATURATION EXTRACT (meq/1)			
	Na	Ca	Mg	K
Kaolinite	0.42	0.50	0.22	0.06
Bentonite	14.52	0.31	0.15	0.46
Ca Montmorillonite	0.58	2.24	0.24	0.28
Soil A	21.65	24.50	11.01	0.38
Soil B	2.14	1.30	0.31	0.09
Soil C	14.35	22.70	5.44	0.62

A saturated paste was prepared and allowed to stand in a moisture controlled room for a few hours before transferring to the consolidation cell (250 g of air-dry clay mineral was used for this purpose). The cations in the extract were determined using the atomic absorption spectrophotometer.

4.2.6 Specific Surface Area

The specific surface of a clay mineral is an extremely important soil parameter used in the estimation of long range repulsive stress R_1. In general, total specific surface area is used to calculate the magnitude of R_1 in a given soil.

The standard method for the determination of specific surface of soil materials is by adsorption of a gas such as nitrogen. Easier methods, which give comparable results, are also available (Hillel, 1971). These are based on retention of a polar organic molecule on the soil surface. Among the methods available, ethylene glycol mono-ethyl ether (EGME) method (McKeague, 1978) seemed to be the most desirable for our purposes. This method is much more rapid than the other available methods.

However, it was noted that the swelling pressure for a montmorillonite clay, with a tactoid structure, has to be calculated for the surface area of the outside of the tactoids. Total specific surface area tends to overestimate the surface area involved in physico-chemical interactions.

Yong and Warkentin (1975) suggested that the surface area measured by nitrogen absorption gives a better estimate of the area involved in the development of physico-chemical forces.

However, the nitrogen absorption method is time consuming, and the results are only approximate. Therefore, it was decided to determine the total specific area from the EGME method and use information available as to the number of particles per tactoid and the total specific surface area determined, to calculate an external specific surface area to be used in repulsive stress calculations.

The total specific surface of each soil was determined on the air-dry soil samples passing a #60 mesh sieve. The soil samples were first treated with two 50 ml batches of 10% H_2O_2, for removal of organic matter.

Thereafter, the soils were saturated with Ca^{+2} by repeated washing with 1N $CaCl_2$. Excess salt was removed by washing with distilled water until the test with $AgNO_3$ was negative. The sample was dried again and ground to pass the #60 mesh sieve.

A sample of 1.1 g of air dried soil sample (in the case of pure clay minerals only 0.3 g were used), was weighed and dried to a constant weight over P_2O_5 in an evacuated dessicator.

The sample prepared in this manner was treated with approximately 3 ml of EGME. This EGME soil slurry was placed in a dessicator over $CaCl_2$ and was equilibrated for about 30 minutes, before evacuating the dessicator. After about 6 hours, the samples were weighed and the dessicator was re-evacuated for about 45 minutes. The weighing was continued every 4-6 hours until a constant weight was reached.

The total specific surface area was calculated as:

$$\text{specific surface area } (m^2/g) = \frac{\text{g of EGME retained}}{0.000286 \times \text{g of soil used}}$$

Because the organic matter content was very low, the H_2O_2 oxidation treatment step was not necessary for the pure clay minerals. Kaolinites and Ca montmorillonite samples contained high concentrations of Ca^{+2} and Mg^{+2} in their exchangeable positions. Therefore, calcium saturation was not considered necessary for these clay soils and subsequently this step was omitted from the adopted procedure. The total specific area values thus obtained (used in calculating repulsive stresses) are tabulated in Table 4.7.

TABLE 4.7

TOTAL SPECIFIC SURFACE AREA FOR CLAY MINERALS AND NATURAL SOILS

SOIL MINERALS	TOTAL SPECIFIC SURFACE AREA (m^2/gm)
Kaolinite	11.80
Bentonite	678.00
Ca Montmorillonite	740.00

4.3 LONG TERM SYNAERISIS CONSOLIDATION EXPERIMENTS

To investigate the influence of the composition of the liquid entering the soil sample on its shrinking or consolidation characteristics, a pneumatically loaded consolidometer was designed. Twelve of these units were built in order to perform several series of tests under a variety of test conditions, to elucidate the following:

- Influence of type and concentration of cations, in the test liquid, on the synaerisis shrinkage phenomenon,
- Influence of soil clay mineral type, exchangeable cation on synaerisis shrinkage, and
- Interaction effects.

A general description of the apparatus used for this purpose and the testing procedure are discussed in the following sections.

4.3.1 Synaerisis Consolidation Apparatus

The synaerisis shrinkage determinations were performed in twelve consolidometers, which were built to suit the requirements of this study. Schematic diagrams of the experimental set-up and of the consolidation apparatus are presented in Figures 4.1 and 4.2.

The apparatus consist of a pneumatically operated loading unit and a consolidation cell surrounded by a leachate reservoir (pressure cell).

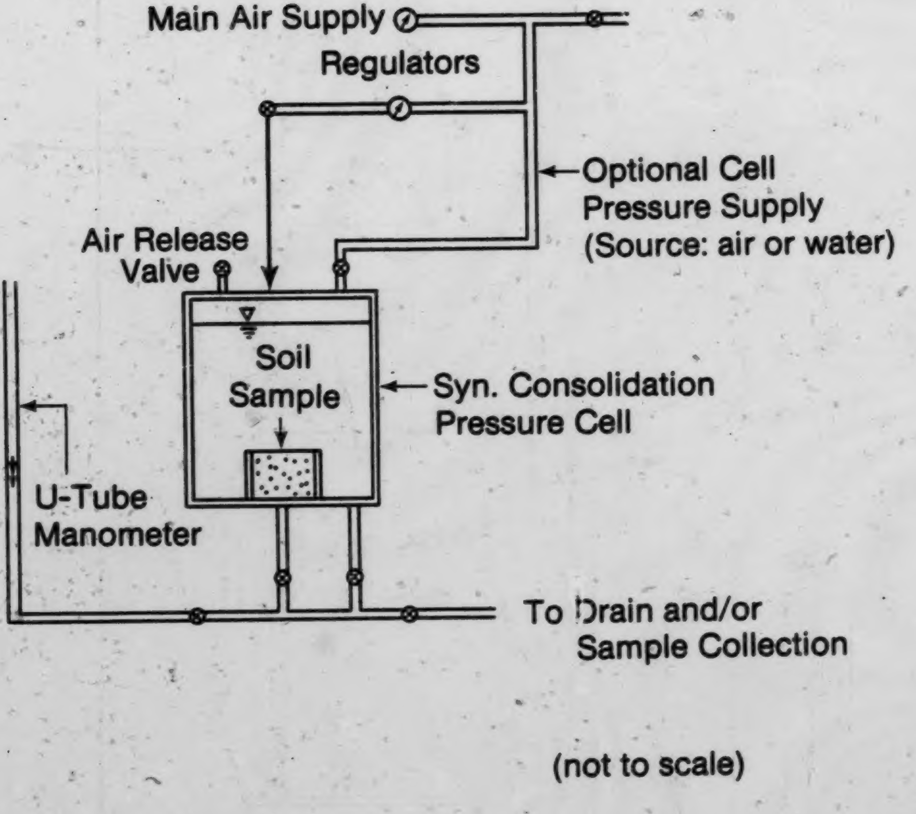
Main Air Supply
Regulators
Optional Cell
Pressure Supply
(Source: air or water)
Air Release
Valve
Soil
Sample
Syn. Consolidation
Pressure Cell
U-Tube
Manometer
To Drain and/or
Sample Collection
(not to scale)

Fig 4.1 SCHEMATIC DIAGRAM OF THE EXPERIMENTAL SETUP

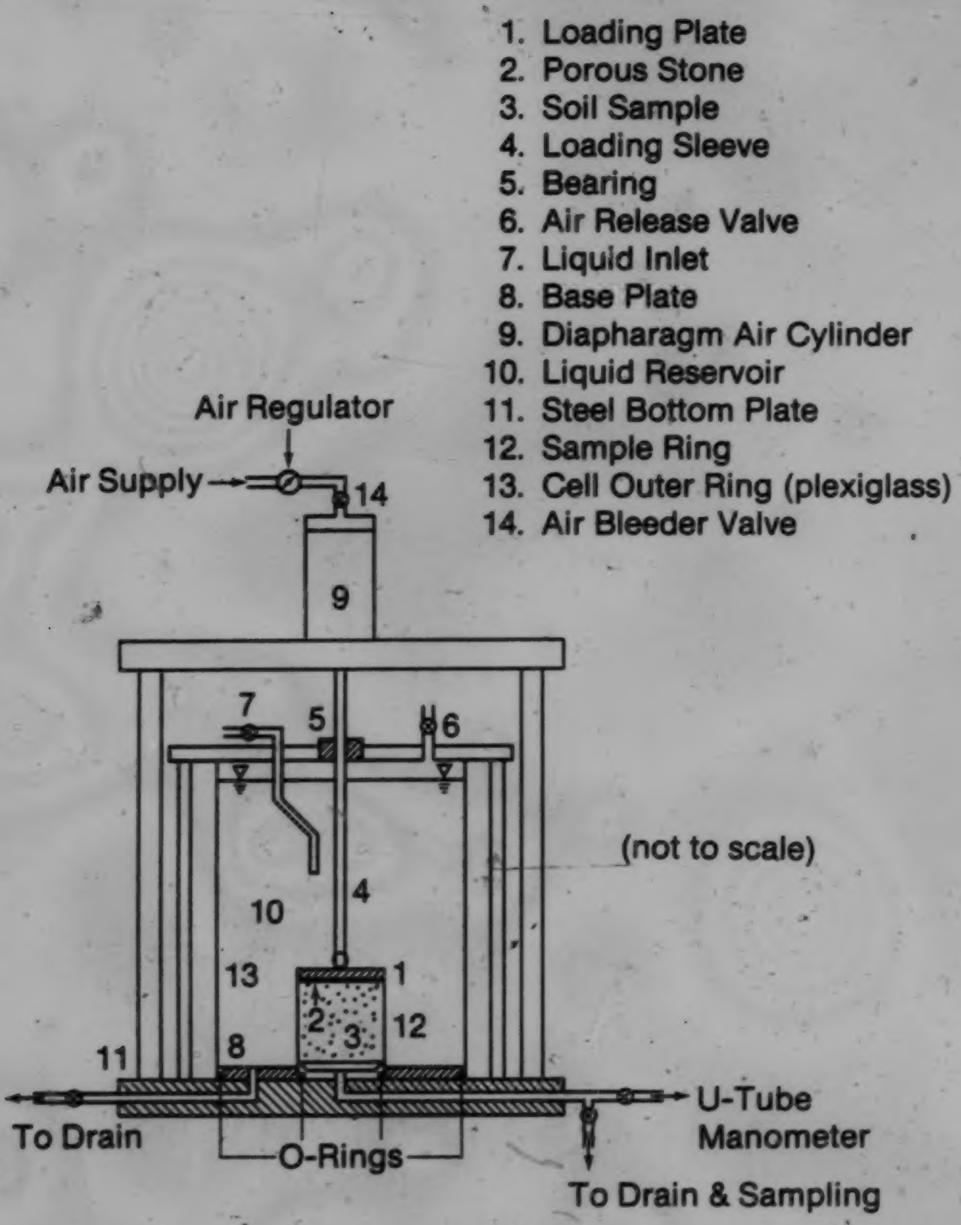
1. Loading Plate
2. Porous Stone
3. Soil Sample
4. Loading Sleeve
5. Bearing
6. Air Release Valve
7. Liquid Inlet
8. Base Plate
9. Diapharagm Air Cylinder
10. Liquid Reservoir
11. Steel Bottom Plate
12. Sample Ring
13. Cell Outer Ring (plexiglass)
14. Air Bleeder Valve
Air Regulator
Air Supply
14
9
7
5
6
(not to scale)
10
4
13
1
2
3
12
11
8
To Drain
U-Tube
Manometer
O-Rings
To Drain & Sampling

Fig 4.2 SCHEMATIC DIAGRAM OF THE SYNAERISIS CONSOLIDATION APPARATUS

4.3.1.1 Loading Unit

Vertical stress to the sample was applied using an external air pressure supply through a pressure regulator and a diaphragm air cylinder, supplied by Belloframs Products Company. The external air supply was maintained at 500 kPa throughout an experimental run. Four regulators were used to supply air at the required pressure to the twelve diaphragm air cylinders. This allowed some flexibility in the application of axial vertical stress to each soil sample.

Since the loading sleeve of the diaphragm air cylinder was found to be shorter than required for our purposes, it was extended by lapping a stainless steel rod of diameter 19 mm and 197 mm long.

In the case of permeability testing, this loading sleeve had to be carefully lapped onto the top plate bearing (see Figure 4.2). The loading piston transmitted vertical stress to the sample, via a 12.7 mm diameter teflon ball. Stainless steel balls, used in the earlier experiments, had to be replaced by teflon balls because of corrosion problems encountered during long term exposure to leachates. The vertical deflections were recorded using a system comprised of a LVDT and a data recording device. An aluminum sleeve was used to connect the LVDT to the loading piston.

All twelve diaphragm air cylinders were tested for sleeve friction, and calibrated. A common calibration curve was developed and used in transferring values obtained at the regulator to the stress values on the sample.

4.3.1.2 Pressure Cell

The test specimen is contained in a stainless steel consolidation cell. Thin (thickness 3 mm) porous discs were provided at the top and bottom of the specimen to allow axial drainage. The top porous disc was fixed to the bottom of the loading cap.

The main components of the pressure cell are described below:

a) base plate - stainless steel, 254 mm outside diameter, 25.4 mm thick, provided with drainage lines and valves for consolidation cell and leachate reservoir drainage.

b) bottom plate - stainless steel, 63.3 mm outside diameter, 12.7 mm thickness, locating drainage grooves, connected to a control drainage outlet was fixed to the base plate by 3 screw down bolts. Two O-ring seals, at the bottom, isolate liquid flow from the consolidating sample which was drained and collected using valve B.

c) sample ring - fixed type, stainless steel, 62.5 mm internal diameter, 76.4 mm deep, wall thickness 3.4 mm. Used an O-ring (fixed on to the outer side of the bottom plate) to hold the ring down to the base plate.

d) Loading cap - 12.7 mm thick stainless steel, provided with holes to assist in liquid drainage. A

0.25 mm gap was maintained between the loading cap and the consolidation ring. A small recess was located in the middle to place a 19 mm diameter teflon ball.

e) Cell outer ring - 137 mm internal diameter, plexiglass, wall thickness 6.3 mm, 203.2 mm high. O-rings used at the top and the bottom to obtain a better seal.

f) Top plate - 12.7 mm thick, 190 mm diameter, stainless steel with an opening of 47 mm diameter in the middle (was fitted with a specially designed bearing, during permeability testing), was provided with liquid inlet and air removal valves (used only during the pressurized mode, in permeability testing).

g) Supporting plate - 25.4 mm thick, 254 mm diameter, stainless steel. The diaphragm air cylinder was fixed to this plate.

4.3.2 Test Procedure

The equipment described in 4.3.1 was found suitable for soil material response studies, in which the stress-deformation response of the soil, subjected to change of a single component of the true effective stress σ^* (either, external stress component or physico-chemical component), could be monitored. The experiments were performed to monitor the shrinkage behavior of a fully saturated soil, prepared by artificial sedimentation of a soil slurry, caused by an increase in true effective stress σ^* (as a result of a decrease in the physico-chemical stress component, (R-A)).

4.3.2.1 Sample Preparation

Consolidometers are usually employed in determining swelling pressures and magnitude of volume change due to absorption or displacement of water. A typical loaded swell test consists of preparing the sample by standard compaction techniques, providing lateral confinement in the consolidation ring, applying vertical stress, allowing free access to surplus water or some other test liquid (Balasubramonion, 1972).

The magnitude of swell and swelling pressure determined from standard techniques was found to be dependent on the sample preparation technique, which would control the soil structure and the initial water content. Initial structure of the soil may differ from one sample to the other, causing experimental variability under test conditions.

Therefore, a pilot testing program was undertaken to develop a reliable sample preparation technique, to prepare soil samples which were representative of the field situation (shear deformation preparation techniques produce a more dispersed structure than occurs in the field). The reproducibility of the initial soil structure, consequently the volume change results, was the main objective for developing the sample preparation technique described below.

Air-dried soil passing #200 sieve (less than 63 micron size) was mixed with de-ionized, de-aired water to a water content exceeding the liquid limit of the soil material. The soil slurry thus prepared was stirred continuously for a few minutes to remove all entrapped air. The vessel containing the slurry was tightly closed and transferred to a moisture controlled cold room and stored for 3 to 5 days. The duration of storage for homogeneous distribution of water throughout the soil sample was found to be a function of the amount of swelling clay mineral present, in our case, the percent of bentonite. The higher the percentage of bentonite, the longer the storage duration. The Na montmorillonite particles required a longer time to hydrate fully and expand to their maximum potential.

If the soil was given enough time to equilibrate with the deionized water in the slurry, the soil structure produced was one of maximum dispersion.

After the equilibration stage, the slurry was taken out from the store room and mixed and stirred for a few minutes, before pouring into the consolidation ring.

To set up the apparatus for a test run, first the porous stones were boiled for about half an hour in deionized water. Two Whatman #42 filter papers were cut to the size of the porous stones and wetted with deionized water. After fixing the bottom plate and sample ring and placing the bottom porous stone and the filter paper, in that order, the soil slurry was slowly poured into the consolidation ring. Stirring the slurry continuously was necessary during placement to avoid entrapment of air within the sample.

Extreme care was taken in lowering the loading cap, containing the top porous stone and the filter paper, because any misalignment would cause tilting and squeezing out of slurry from one side and/or subsequent sticking (caused by the loading cap coming in contact with the interior of the consolidation ring). Keeping 1/4 mm clearance between the loading cap and the consolidation ring and applying high vacuum grease, both on the outside edge of the loading cap and the inside of the consolidation ring, was necessary. The amount of slurry poured into each cell was such that the loading cap extended at least 6.5 mm above the top of the consolidation ring. This was essential for the proper placement of the loading cap.

Once loaded, the consolidation ring containing the soil specimen was submerged in about 178 mm of deaired, deionized water. The soil sample was allowed to consolidate under the load of the loading cap (weighed 354 grams) and the water head provided by the reservoir water. The reservoir held about 3 L of liquid and was maintained at the same height throughout the entire experimental run.

The system was kept in a temperature (20°C) controlled room for several days, before, external loading using the diaphragm air cylinders (or Bellofram), was started. As indicated by theoretical considerations, presented in

Chapter three, changes in temperature could influence the volume change behavior of clay soils. Therefore, it was necessary to perform these experiments in a temperature controlled environment.

Consolidation, until equilibrium, under the initial smaller load, was found essential to prevent slurry leaking as a result of the sudden application of the initial external load from the air cylinders. The deflection produced by the loading cap and the liquid (called "zero load" from hereon), was measured before setting up the apparatus for load application from the air cylinders.

Initial stress applied from the air cylinders was 10 kPa. Most of the samples produced their largest deflection at this "zero load". Some amount of slurry leaked out from several soil samples, during this load increment. However, the amount leaked out was found to be very small and was accounted for during the analysis of conventional consolidation and synaerisis consolidation data. However, in the first experimental series, some difficulty arose when the loading cap stuck during the process of consolidation for sample preparation in 3 of the 12 cells. These cells were abandoned and they were not tested for synaerisis consolidation. The problem arose when larger soil particles (the soil used in this set of experiments was the portion passing #50 sieve), while being squeezed out, lodged in the gap between the loading cap and the consolidation cell. This effectively prevented the downward movement of the loading cap during load increments. Subsequently, the soil fraction passing #200 sieve was used, for the rest of the experimental runs.

The physico-chemical forces in a soil system are manifested among the clay sized particles. Clay minerals in the soil would control the synaerisis shrinkage behavior of a soil sample. The soil fraction passing #200 sieve would contain more clay sized particles, as a percentage, and should show clearly the effect of changing physico-chemical forces within the soil sample.

The test specimen was allowed to consolidate under a number of successive increments of vertical pressure, each pressure increment being maintained constant until at least the primary consolidation was fully completed. The successive vertical pressures employed were 10, 20, 40, and 80 kPa respectively.

The soil specimen was maintained at 80 kpa for several days even after primary consolidation had been completed. This pressure was calculated to be equal to the surcharge pressure created at the bottom of a 10 m deep municipal solid waste landfill. Experience showed this to be necessary, to establish the complete time settlement (both primary and secondary) curve at this pressure.

4.3.2.2 Synaerisis Consolidation Measurements

When the deformation became negligibly small (as indicated in a time settlement curve), the sample was considered ready for leachate treatment. The deionized water was drained out and the entire pressure cell was first flushed with the synthetic leachate. Thereafter, the drainage valve "A" was closed and the reservoir filled back to the initial liquid height. The 178 mm hydraulic head provided was found sufficient to produce a response within a practical time frame.

The axial deflection caused by volume shrinkage, as a result of leachate addition, was continuously measured using the LVDT's, and the data recording system. In some of the experimental runs, the liquid squeezed out from the samples and/or liquid "breaking through" was collected and analyzed for cations, Ca^{+2}, Na^{+}. This gave an indication of how fast the permeating leachate travelled across the soil sample.

Axial pressure was maintained at 80 kPa, during the leachate permeation and interaction stage. When no vertical deflection was being recorded, the clay soil-leachate interaction was considered essentially complete, and a further stress increment to 160 kPa was provided.

The soil was allowed to consolidate, at this pressure, until the completion of primary consolidation. Then the leachate was drained out, the pressure was released and the sample removed from the consolidation cell. Extreme care was taken in extruding the sample from the consolidation cell without disturbing the sample and losing soil. The thickness of the sample was measured. The sample was weighed and transferred to an oven maintained at 103°C. After 24 hours the oven dry weight was measured and used in calculating the final water content.

Samples selected for electron microscopic studies were put in freezer bags immediately after removal, and transferred to a moisture controlled cold room. Special care was taken not to disturb the sample in any way during ejection and removal of soil specimen from the consolidation ring. This was essential because any disturbance would change the soil microstructure, and would invalidate findings of the microstructural investigation.

4.3.2.3 Sensitivity of the Synaerisis Consolidation Test Procedure

Scientific data may contain "noise", ie: the data may include disturbances caused by measurement and incomplete control of the experimental environment. The confounding effects of experimental error are greatly reduced by proper experimental design. An estimation of experimental error could be done by including replicates in an experimental design or by performing independent error estimates.

Series #4 experiments were performed specifically to obtain the error estimates for synaerisis consolidation experiments. Table 4.8 summarizes the shrinkage results obtained during this experimental run. Based on the series #3 experiments, 2 types of soil mixtures and electrolyte solutions were used.

Soils used were:

- 6 mixtures of 15% bentonite and 85% kaolinite,
- 6 mixtures of 75% bentonite and 25% kaolinite.

TABLE 4.8

Sensitivity of Shrinkage Experiment

Soil (% bentonite **)	Ca Concentration * (moles/L)	Percent Strain
15	0.045	4.30
15	0.045	4.28
15	0.045	4.13
15	0.018	3.14
15	0.018	3.14
15	0.018	2.82
75	0.045	25.67
75	0.045	21.47
75	0.045	24.80
75	0.045	9.01
75	0.018	20.29
75	0.018	11.32

* Na and HAc conc. were maintained at 0.017 M and 250 mg/l, respectively.

** bentonite was mixed with kaolin.

2 liquid types, both consisting of a base (0.017M Na^+ and 250 mg/L HAc) and either 0.018M Ca^{+2} or 0.045M Ca^{+2} were used with each soil type to provide 4 combinations with 3 cells at each.

The results indicate the high reproducibility of shrinkage results at low percent bentonite (15%) content. Percent strain with 0.045M Ca^{+2} showed a mean value of 4.24 and a standard deviation of 0.093. The error, as a percentage, was less than 2, with the lower Ca^{+2} (0.017M), whereas, the error was less than 5% with the higher (0.045M) Ca solution.

However, results with higher bentonite percentages (75%) showed a larger variation with the weaker electrolye. With the stronger leachate, percentage error variation was 9.1%, whereas with the weaker solution, it was as high as 44%. However, cell C-4.11 charged with Ca^{+2} = 0.018M showed signs of corrosion taking place within the system. This seems to have contributed to the observed higher shrinkage and resulting higher error variances.

However, the analysis demonstrate the reliability of shrinkage results at lower percent bentonite levels. Most of the swelling in natural soils falls into this category. Careful control is necessary to obtain reproducible results with higher bentonite percentages or highly swelling soils.

4.4 LONG TERM DIRECT PERMEABILITY TESTS

In Chapter 3, the general effect of electrolytes to increase the permeability across a given soil sample was noted. Although several mechanisms have been identified by others, at least in some cases, volume shrinkage and minute crack formation were likely the chief causes of increased permeability.

The consolidation results were used in calculating the k (hydraulic conductivity) values for soil samples prior to and after leachate permeation. Contrary to the popular belief, the results (to be discussed in the next chapter) would not provide overwhelming evidence of a large increase in hydraulic conductivity, as a result of permeation with solutions containing high electrolyte concentrations. In general, calculated k values after leachate permeation, at an axial pressure of 160 kPa, showed a slight decrease over the deionized water k values (calculated from 80 kPa consolidation results).

However, analysis of experimental procedures in each situation is essential before drawing conclusions from any experimental result. As discussed in earlier chapters, experimental procedures definitely contribute to the outcome. Therefore, to verify the results obtained from consolidation and subsequent calculation of k values, direct permeability tests were performed on a limited number of soil samples.

4.4.1 Permeability Testing Apparatus

Because these experiments were performed to verify the results obtained from the "indirect method" by calculation, it was essential to perform permeability tests on samples prepared in an identical manner.

The same consolidation equipment was used for this purpose with minor modifications. A higher hydraulic gradient, adequate to force liquid through the soil sample and to obtain valid results within a practical time frame was necessary. The consolidation apparatus was modified to accommodate such an arrangement.

Performing direct permeability tests at high hydraulic gradients required complete sealing of the leachate reservoir. This allowed pressurizing the reservoir with air or liquid pressure. The stainless steel loading piston was carefully lapped into a bearing located in the middle of the top plate. Proper vertical alignment of the bearing and the sleeve opening was essential in minimizing the side friction caused during the piston travel through the bearing opening. However, it was necessary to recalibrate the loading unit to remove the errors caused by frictional resistance. Air outlet and liquid intake valves were fitted on to the top plate, to help in providing an access to an external liquid reservoir and to allow for air release, respectively.

Connecting the pressure cell to an external air pressure system and providing a very high hydraulic gradient was feasible. However, this was not attempted for the following reasons:

- very high hydraulic gradients are not representative of the field situation,
- very high gradients would cause micro structures to change and subsequent channeling to occur,
- external pressure may create unnecessary leaking problems within the system; in particular, leaking between the consolidation cell, bottom plate and base plate would cause short circuiting and permeability increases.

By locating an overhead leachate reservoir above the experimental set up, it was possible to apply a hydraulic head of about 1.5 m, which corresponds to a hydraulic gradient of about 60 (for an average sample thickness of 25 mm). Permeability testing with very low hydraulic gradients (about 10) were also tried. However, in calculating hydraulic conductivity from such permeability results, liquid drainage caused by synaerisis consolidation (during leachate permeation) has to be taken into consideration.

4.4.2 Permeability Testing Procedure

Only 4 cells were modified and 4 soils were used in direct permeability measurements. Two cells, in which native soil passing #40 sieve were artificially sedimented, had to be abandoned during the sample preparation phase of the experiment. Improper alignment caused one of the loading caps to be tilted, preventing further downward movement during load application. The slurry squeezed out during initial loading of the second cell and resulting higher friction caused by escaping larger particles (the sample contained sand and silt sized particles) slowed the downward movement of the loading cap. Since the frictional resistance caused by this action could not be evaluated correctly, this cell too was abandoned. By resorting to using pressure transducers to measure stresses on the sample, this limitations could be overcome in future experimental studies. Subsequently, these two cells were reloaded with kaolin/bentonite soil slurries and prepared for low, hydraulic gradient(about 10) permeability studies, with concentrated salt brine as the permeant.

Calibration curves, developed using load tests, were used throughout the experimental program to assess the magnitude of externally applied effective stress.

The soil samples, for direct permeability testing, were prepared by the same procedure described under synaerisis consolidation. Once samples were fully consolidated at 80 kPa, direct permeability tests with deionized water were performed on these samples. Before taking burette measurements, it was absolutely essential to make sure that no air bubbles were visible in the outlet tube. The presence of entrapped air bubbles could incorporate unaccounted errors into the determined hydraulic conductivity values. Therefore, this problem was minimized by flushing the outlet valves and tubes through the base, at the beginning of the experiment, with deaired water.

Since the head loss associated with flow through the soil sample was minimal, these experiments can be categorized as constant head, fixed membrane permeability tests. The tests were performed for a long enough time to yield a constant value for k.

Once permeability testing with deionized water was terminated, drainage valve "A" was opened and deionized water was allowed to drain. Simultaneously, the overhead reservoir was filled with the test liquid and was allowed to drain into the leachate reservoir and displace the deionized water. Draining of the deionized water was considered completed, once a conductivity reading on the draining water shows the conductivity to be the same as the leachate.

The leachate reduced the (R-A) component of the effective stress and increased the true effective stress, resulting in a volume decrease. The time dependent decrease in volume was monitored continuously. The drainage through the sample was monitored using a 10 mL burette.

The liquid collected in the burette, immediately after liquid change, would consist of both seepage water and water squeezed out during synaerisis consolidation. Therefore, during the first few days of leaching,the flow rates entering and exiting the specimen would not be equal. The outflow exceeded the inflow by the amount the specimen pore volume is reduced by synaerisis consolidation, or the amount of liquid squeezed out during synaerisis consolidation. If the permeability tests were performed under very high hydraulic gradients, the contribution to the flow from the second source could be considered negligible. However, low hydraulic gradients were adopted in this research program. Therefore, the total flow was smaller and the contribution from consolidation drainage was substantial, and could not be ignored.

Therefore, valid constant-head permeability tests could only be performed after consolidation, due to synaerisis, has been completed. However, burette readings were taken regularly and corresponding drainage flow was calculated throughout the entire period of liquid permeation. Time dependent variation of hydraulic conductivity, thus monitored, and the determination of volume shrinkage during this time period, would help in assesing the contribution from synaerisis consolidation to the amount of liquid collected in the burette during a given time period.

Volume deflections, inflow and outflow hydraulic heads relevant for each burette readings, were recorded. In addition, leachate samples from the drainage line were collected (whenever possible) and analyzed for calcium and sodium. Increases in the cation concentration in the drainage liquid were considered as an indication of leachate breakthrough across the sample.

Once the shrinkage due to synaerisis consolidation was diminished to low levels, the flow rate across the sample reached a constant value. Time taken to approach this point varies with different soil samples and different liquids used in the permeability study; 2-3 weeks was necessary for the two bentonite modified native soils, tested with standard leachate. Several permeability measurements were made, and calculated hydraulic conductivities checked for variability, before accepting a value for true hydraulic conductivity (k) for the soil with leachate. This k value is directly comparable with the value obtained for deionized water.

Later in the program, direct permeability tests with brine as the permeant were performed. The synaerisis consolidation apparatus was fitted with a graduated burette to monitor the liquid outflow. Although, the consolidation apparatus (without provision for pressurizing the leachate reservoir) could be used as a permeameter, a major disadvantage was the inability to provide a high hydraulic gradient.

4.5 SEDIMENTATION EXPERIMENTS

In the synaerisis consolidation experiments, measurement of cations in the permeant before and after passage through the soil indicated that, when the leachate was used, ion exchange of divalent cation for monovalent cations,(generally sodium) occurred. The ion exchange caused reduction of long range repulsion and subsequent synaerisis shrinkage. In a soil suspension, exchange of divalent cations for monovalent cations, causes soil to flocculate.

Although the behavior of a soil suspension is not the same as that of a compacted material, a similar effect could be observed on interparticle forces in both situations. Preliminary studies showed that the effect of cations would be to create a smaller sedimented volume after flocculation. Therefore, it was decided to study the effect of simulated leachates on sediment volumes and verify the possibility of developing a simple procedure involving the flocculation concept. Furthermore, the possibility of using the simpler, much faster experiment as an index test needed to be explored.

4.5.1 Development Of The Experimental Procedure

Holtz and Gibbs (1956) first proposed the free swell test as an index test to identify swelling soils. This test is performed by slowly pouring 10 mL of dry soil, passing the 425 micron sieve, into a 100 mL graduated cylinder filled with distilled water. The volume of the settled and swelled soil is read after 24 hours.

The percentage free swell (%FS) is then determined from the expression:

$$\%FS = \frac{V_f - V_i}{V_i} \times 100 \qquad 4.1$$

where,

V_f = final volume,

V_i = initial dry volume (10 mL).

According to this test, bentonite (sodium montmorillonite) may swell from 1200 to 2000 percent, whereas kaolinite and illite swell from 30 to 80 percent (Singh, 1976). Holtz and Gibbs (1956) suggested that, soils having free-swell values as low as 100 percent may undergo considerable volume changes when wetted under light loading and should, therefore, be viewed with caution.

During preliminary testing, it was decided to eliminate the probable errors (in the test procedure) due to the initial placement density of the soil (initial volume observation). Singh (1976) reported an alternative method, in which a weighed quantity of oven-dried soil is placed in a sintered funnel and allowed to absorb benzene. Thereafter, it was allowed to absorb water in place of benzene. The difference between the respective volumes of water and benzene absorbed represents the swelling. The results obtained are independent of pore volumes because absorbed benzene measures the pore volume and water measures the absorption required to fill the pore volume and cause swelling.

Benzene is known to destroy the crystal structure of the clay mineral. Therefore, use of benzene to measure a soil parameter without fully understanding the consequences of benzene-clay interaction is questionable. Although this technique was tried during preliminary testing, it was not considered a viable alternative.

The main aim in performing sedimentation experiments is to understand the flocculation behavior of different soils in different electrolyte systems. The experimental procedure should be able to provide reproducible, reliable results. The information that one should be able to extract must include sediment volumes under varying flocculating conditions for competing soil types. The sedimented soil volumes should be evaluated with respect to each flocculating condition and be used in developing a new parameter, FSDP, for each soil and environment condition.

To achieve these objectives, the following procedure, which will be referred to as the "sedimentation experimental procedure" from here onwards, was adopted.

The selected soil sample was air-dried and sieved. Soil passing #200 sieve (62 micron size) was collected. 10 g of the collected soil was slowly tipped into a 100 mL graduated cylinder containing a test liquid. The rate of addition of the dry soil depends upon the rate at which the soil particles are hydrating at the air water interface. High swelling soil, particularly the soils containing Na montmorillonites as the clay mineral, take a longer time to fully hydrate and form flocs that eventually settle at the bottom of the graduated cylinder. At higher electrolyte concentrations, no matter what kind of clay mineral was present, the flocs formed faster and permitted speedy addition of soil.

The total time required to add 10 g of soil to various electrolytes ranged between 10 to 90 minutes. Therefore, 4 to 5 sedimentation tests were performed concurrently, and to minimize the waiting period between each "drop". Each drop contained 100-300 mg of soil; the higher the clay content, smaller would be the drop size.

Immediately after adding the soil, sediment volume readings were taken. The settled soil undergoes self weight consolidation and the amount of volume decrease at a given time is always a function of initial sediment volume, except when the sedimenting liquid was either deionized water or a liquid of very low electrolyte content. Self weight consolidation occurs as a result of one way drainage of pore fluid.

Soil sediment volume was recorded for at least seven days or until no change in volume, with time, was observed. For most of the soils and electrolyte solutions, 36 hours was considered adequate to provide a reasonable "equilibrium soil volume".

If the readings were taken for an extended period of time, it was found necessary to close the graduated cylinder and prevent evaporation. Evaporation increases the electrolyte concentration and would contribute to further consolidation because of synaerisis.

4.5.1.1 Reproducibility of Sedimentation Test Results

As noted in section 4.3.2.3, it is essential to obtain an estimate of the experimental error when dealing with experimental results. Reproducibility of the experimental procedure and the test results needed to be checked, before accepting the sedimentation test as a valid experiment.

To generate results needed for an estimate of the variance, 9 trials were performed with 25,000 mg/L NaCl as the test liquid. Test soil samples were made up of 10 g each of bentonite/soil C (at a ratio of 1:2) mixtures. The final sediment volumes are report[illegible]n Table 4.9.

The results from 9 trials gave a mean value of 16.02 and a standard deviation of 0.21, for a percentage of error of only 1.3%. These results indicate the very high reproducibility of the test results. However, considering the possibility of performing 4 or 5 trials at one time, it was decided to include at least 2 replicates (particularly for the higher bentonite soils) and take the average value as the final sediment volume. Therefore, throughout the program the average of 2 readings, (sometimes 3) was taken as the equilibrium sediment volume.

TABLE 4.9

Sedimentation (Replication) Results

Trial #	Sedimentation Volume (ml)
1	16.0
2	16.0
3	16.0
4	16.0
5	15.6
6	16.4
7	16.2
8	16.0
9	16.0

Sedimentation Liquid - 25,000 ppm, NaCl

Soil - bentonite/soil C = 1:2

4.6 MICROSTRUCTURE STUDIES

The study of the various levels of macro and micro-structure is accepted as essential to a basic understanding of the engineering behavior of clay soils (Sides & Barden, 1971). In order to support studies of the mechanisms of soil shrinkage, and changes in hydraulic conductivity, observation of microstructural features before and after leachate permeation, was necessary for at least a limited number of samples.

4.6.1 Scanning Electron Microscope

Microstructural features, such as orientations of particles and the presence of micro cracks are particularly important in soft, normally consolidated (most of the samples tested during this research program fall into this category) clays, which contain very few macro fabric features.

Electron microscopy, because of its high resolving power, is a suitable technique for the detailed identification of clay sized particles (Pusch, 1970). Using a stereo-scan, electron-microscope, it is possible to study the different faces of a soil sample over a complete range of magnification from 20 times to 50,000 times. The large depth of focus provides a strong, three-dimensional picture, which enables rough, unpolished surfaces of both coarse and fine particles to be observed. A good review of the application of the electron microscopy, in studying the fabric of fine grained soils, was provided by Gillot (1969). The possibility of examining relatively large samples (about 10 mm diameter) at low and at high magnifications, when areas of major interests have been located, and the relative simplicity of sample preparation, are mentioned as major advantages of the scanning electron microscope.

For this purpose, a Cambridge Stereoscan S4 electron microscope, available in the Entomology Department, which has a magnification range of 14 to 200,000 times, was used.

4.6.2 Specimen Preparation And Electron Microscopic Observation

Several soil samples, from the series #5 experiments, were removed from the consolidation cells and stored in a humid cold room. These include 3 leachate attacked samples and a "control" soil sample.

The electron microscope technique involves the direct reflection of electrons from the surface of the sample. Therefore, the quality of the images produced would be highly dependent on appropriate and careful preparation of the specimens. Poor preparation can lead to disturbance of otherwise undisturbed soil fabrics and to poor images.

It is not possible to analyse the fabric of wet soil samples by electron microscopy. The water must be replaced or removed. Techniques available for this purpose are, air drying, critical point drying and freeze drying techniques. The air-drying technique produces considerable shrinkage.

The fabric of the selected soil samples (having a high water content) would be significantly affected in such a situation. Shrinkage caused by surface tension forces (caused by drying) and moisture migration (caused by freezing) can be reduced by using either critical point drying or freeze-drying techniques.

In this investigation, the less complex, much faster freeze-drying technique was employed in preparing specimens for the scanning electron microscope. Tovey and Wong (1980) reported that freeze drying caused negligible shrinkage of specimens, and was always less than 0.5%.

A small sample of the soil was frozen and resulting ice removed by sublimation under vacuum coolant in the freezing step. The prepared specimens, once mounted and viewed on the electron microscope, showed the presence of a large number of minute cracks. Since it was suspected that the cracks were formed during the sample preparation, specimens from the same soil samples were prepared again using Freon as the coolant.

Higher cooling rates were necessary to minimize shrinkage. Freon increased the rate of cooling. During the microscopic analysis, the sample size and the shape factor (volume/total surface area) were kept to a minimum. Provision of optimum cooling conditions reduced the number and the size of micro cracks visible on the soil samples.

To further study the cause of the micro cracks, an elemental analysis was performed in the vicinity of the cracks. The objective was to see whether the elemental composition in and around the cracks was significantly different from the composition in the bulk material. If preferential liquid movement has taken place along macrovoids, the elemental analysis is capable of indicating the location of such voids. Higher cation concentrations were expected in the vicinity of these macrovoids.

5. PRESENTATION, ANALYSIS AND DISCUSSION OF EXPERIMENTAL RESULTS

5.1 INTRODUCTION

The basic aim of this research was to understand the mechanisms controlling the permeability changes that would occur as a result of leachate interaction. The literature review identified volume shrinkage as a major contributor to increased permeability, both in the field and in conventional laboratory permeability testing ("breakthrough" phenomenon observed by several investigators).

Therefore, the main thrust of the experimental program was directed at studying the contribution of leachate to volume shrinkage. This was accomplished by direct measurement of volume change during synaerisis consolidation experiments and the equilibrium sedimented volumes in sedimentation experiments.

Consolidation results provided the means of indirect determination of hydraulic conductivity (k) changes caused by leachate interaction. These results are presented and discussed with some of the results obtained by direct permeability testing.

The investigations directed at studying the changes in microstructural features, caused by leachate interaction, contribute much in understanding observed permeability changes. The results obtained from the electron microscopic investigation are presented at the end of this chapter.

System chemistry data, Atterberg limits, mineralogical data and exchangeable cation information will be presented as necessary to explain observed volume shrinkage and/or permeability behavior.

5.2 SYNAERISIS CONSOLIDATION STUDIES

Synaerisis consolidation characteristics of the clay soils were obtained from the experimental procedure explained in Section 4.3.3. The procedure allowed testing of different soil samples, containing varying amounts of different clay minerals, with various liquids at desired surcharge pressures. Therefore, experiments were designed to identify and quantify the effect of the following factors on synaerisis shrinkage phenomenon:

- leachate characteristics,
- clay minerals (including the type and amount) in the soil,
- type and percentage of exchangeable cations within the soil,
- axial pressure and the initial void ratio.

The bulk of the experiments were performed with soil samples made up of mixtures of commercially available clay minerals. Three clay minerals, Na montmorillonite (Wyoming bentonite contains 85% Na montmorillonite and 15% Illite), kaolinite and Ca montmorillonite were selected for this purpose.

In addition, experiments were conducted to investigate the effect of leachates on three local soil~~s~~ and bentonite modified soils.

5.2.1 Relationship With Leachate Characteristics

A basic aim of the experimental program was to identify the leachate constituents that are directly responsible for causing volumetric changes. The literature survey identified the main cause of synaeresis shrinkage as the changes brought about by leachate in interparticle physico-chemical forces. The main contribution is from the inorganic cations in the leachate.

Leachate contains a large number of mono- and di-valent cations, major contributors being Mg^{+2}, Ca^{+2}, Fe^{+2}, Na^{+} and K^{+} ions. Because of the limited number of experiments that could be performed at one time, it was necessary to select representative mono- and di-valent cations to be used in volume shrinkage determinations.

Ca^{+2} and Mg^{+2} are the most common divalent cations present in leachates, followed closely by Fe^{+2}. Fe^{+2} was not considered because of the corrosion problems foreseen.

A series of sedimentation experiments (discussed in detail in Section 5.3) were performed to compare the effect of Mg^{+2}, Ca^{+2}, Na^{+} and K^{+} on interparticle forces. Mg^{+2} and Ca^{+2} behave in an almost identical manner and gave similar results in a sedimentation test. As indicated by double layer theory, whatever discrepancy that exists can be attributed to the differences in hydrated ion radius, with Mg^{+2} exhibiting a slightly larger hydration ion radius than Ca^{+2}.

However, in selecting between Ca^{+2} and Mg^{+2}, some other factors were also taken into account. They include:

- the predominant cation type in natural clay soils of Alberta is Ca^{+2},
- Ca montmorillonite was commercially available (but, Mg montmorillonite was not), which was essential in performing series #4 experiment.

Na^{+} and K^{+}, are the most common monovalent cations in leachate. However, it is well known that K^{+} interacts with montmorillonites in a rather unpredictable way (see Appendix B1 - clay mineral properties) and would not follow the double layer theory. K^{+} is capable of reducing the interlayer expanding capacity of the clay mineral montmorillonite, and was not considered the best example of a monovalent cation in clay electrolyte water system (especially a system containing montmorillonite).

Acetic acid is by far the major organic acid in the organic fraction of the leachate. Therefore, acetic acid was considered representative of the organics found in MSW landfill leachates and used as the third factor in this research.

Both $(SO_4)^{-2}$ and Cl^{-} are abundant in leachates. Sulphates are capable of causing charge reversal in certain clay systems and would participate actively in clay leachate interactions. Although Cl^{-} adsorption is possible at very low pH and with kaolinites (existence of positively charged edge surfaces are well known), chlorides are considered relatively inactive. Therefore, $CaCl_2$ and NaCl were used throughout the study (instead of Na_2SO_4 and $CaSO_4$).

Experimental Series #1

The first set of experiments were performed to identify the main effects of the factors, inorganic cations Na^+, Ca^{+2} and acetic acid together with their interaction effect. The upper and lower settings for each factor was chosen based on the reported maximum and minimum concentrations of monovalent, divalent cations and organic acids (Table 3.1).

These three factors were used in a 2 level factorial design (2^3) consisting of 12 trials. Three replicates and a "control" (de-ionized water used in place of the leachate) were also included with the 8 experimental runs consisting of the two settings of the three factors.

Table 5.1 summarizes the information pertaining to this run. Initially, all 12 cells were loaded with the same soil (a clay mineral mixture containing 67% bentonite and 33% kaolinite) and were consolidated to the required 80 kPa pressure under similar controlled conditions. Two cells failed during this preparation phase of the experiment. At this point, it was decided to sacrifice the "control" experiment and change centre point replication to corner point replication (Davies, 1979) to salvage a complete design from the rest of the cells. However, after charging the 10 cells with 8 combinations of liquid, one cell failed due to the reasons cited in Section 4.2. Table 5.2 provides a summary of the synaerisis consolidation results obtained

TABLE 5.1

SERIES #1 EXPERIMENTAL DESIGN

a) DESIGN MATRIX

CELL #	MAIN EFFECTS AND INTERACTIONS						
	A	B	C	AB	BC	CA	ABC
C 1.1				CONTROL			
C 1.2	-	-	-	+	+	+	-
C 1.3	-	-	+	+	-	-	+
C 1.4	-	+	+	-	+	-	+
C 1.5	-	+	-	-	-	+	+
C 1.6	+	-	-	-	+	-	+
C 1.7	+	-	+	-	-	+	-
C 1.8	+	+	-	+	-	-	-
C 1.9	+	+	+	+	+	+	+
C 1.10	-	+	-	-	-	+	+
C 1.11	-	+	-	-	-	+	+
C 1.12	-	+	-	-	-	+	+

b) FACTORS AND LEVELS

FACTORS			LEVELS		
			(-)	(0)	(+)
A - Na	conc.	(moles/l)	0.017	0.180	0.342
B - Ca	conc.	(moles/l)	0.009	0.072	0.135
C - HAc	conc.	(mg/l)	1,000	10,500	20,000

c) SOIL

Bentonite = 67 % (i.e. Na mont. = 57 %, Illite = 10 %)

Kaolinite = 33 %

TABLE 5.2

SUMMARY OF SYNAERISIS CONSOLIDATION RESULTS - SERIES #1

CELL #	SOIL TYPE	DRY SOIL WEIGHT (grams)	AMOUNT Na MONT. (grams)	INITIAL HEIGHT (Before Leachate Addition) (mm)	Na	Ca	HAc	TOTAL DEFLECTION (Due to Leachate Addition) (mm)	PERCENT STRAIN (%)
C 1.1	Bentonite = 67 % Kaolinite = 33 % (passing #50 sieve)	(EXPERIMENT FAILED)			CONTROL			—	—
C 1.2		32.17	18.34	28.54	-1	-1	-1	1.62	5.68
C 1.3		32.12	18.31	27.96	-1	-1	+1	5.26	18.81
C 1.4		(EXPERIMENT FAILED)			-1	+1	+1	—	—
C 1.5		31.68	18.06	24.83	-1	+1	-1	5.34	21.50
C 1.6		32.39	18.46	26.59	+1	-1	-1	4.12	15.50
C 1.7		31.45	17.93	25.28	+1	-1	+1	3.74	14.80
C 1.8		31.90	18.18	26.30	+1	+1	-1	5.70	21.67
C 1.9		31.81	18.13	24.53	+1	+1	+1	5.27	21.48
C 1.10		31.73	18.09	25.31	-1	+1	-1	5.49	21.70
C 1.11		(EXPERIMENT FAILED)			-1	+1	-1	—	—
C 1.12		32.04	18.26	23.91	-1	+1	-1	5.24	21.92

in the experiment series #1.

The experimental procedure provided the details of consolidation behavior of all the samples in the following stages:

- consolidation of a slurry from a minimum, up to a 80 kPa externally applied axial pressure,
- synaerisis shrinkage due to leachate interaction at a constant external pressure, σ, of 80 kPa,
- consolidation at the load increment, 160 kPa.

The consolidation data thus obtained is used in preparing the settlement vs √time curves for all nine consolidation cells. The summary of synaerisis shrinkage results are included in Table 5.2. Calculated void ratio-effective stress values are tabulated in appendix F as Tables F1.1 to F1.9.

Two of the settlement versus √time plots are shown in Figures 5.1 and 5.2 and the rest are included in Appendix E as Figures E1.1 to E1.3. In Figure 5.1 and Figure 5.2 consolidation settlements caused by vertical pressure increment 80 kPa, true effective stress increment caused by leachate and the load increment 160 kPa, are combined together and plotted against √time. These two plots are selected for further discussion mainly because they were charged with the lowest and highest Ca^{++} concentrations (keeping Na^{+} and HAc constant at their lower levels) and showed the synaerisis shrinkage phenomenon caused by cations.

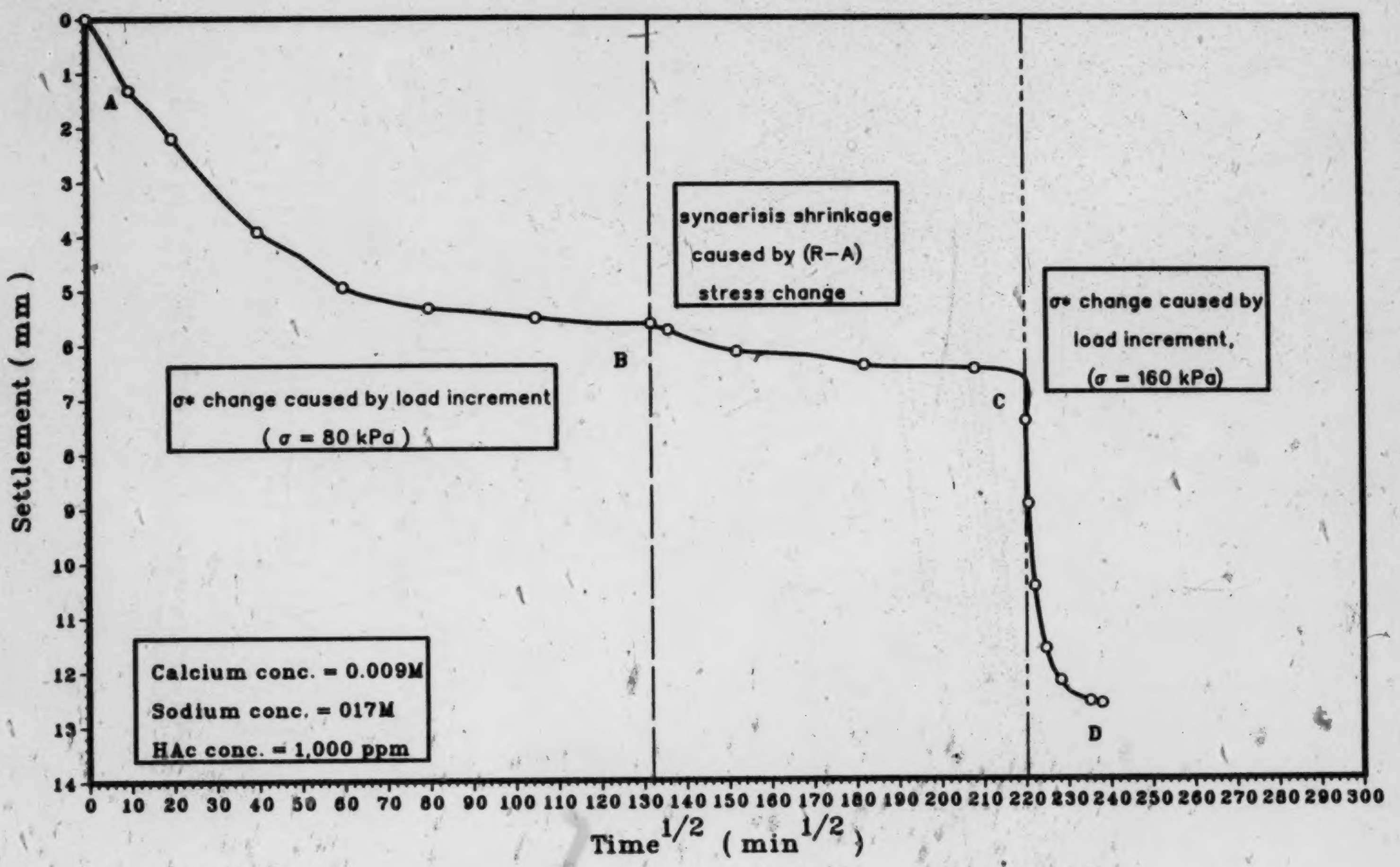

Figure 5.1 Time Settlement Curve for cell C–1.2

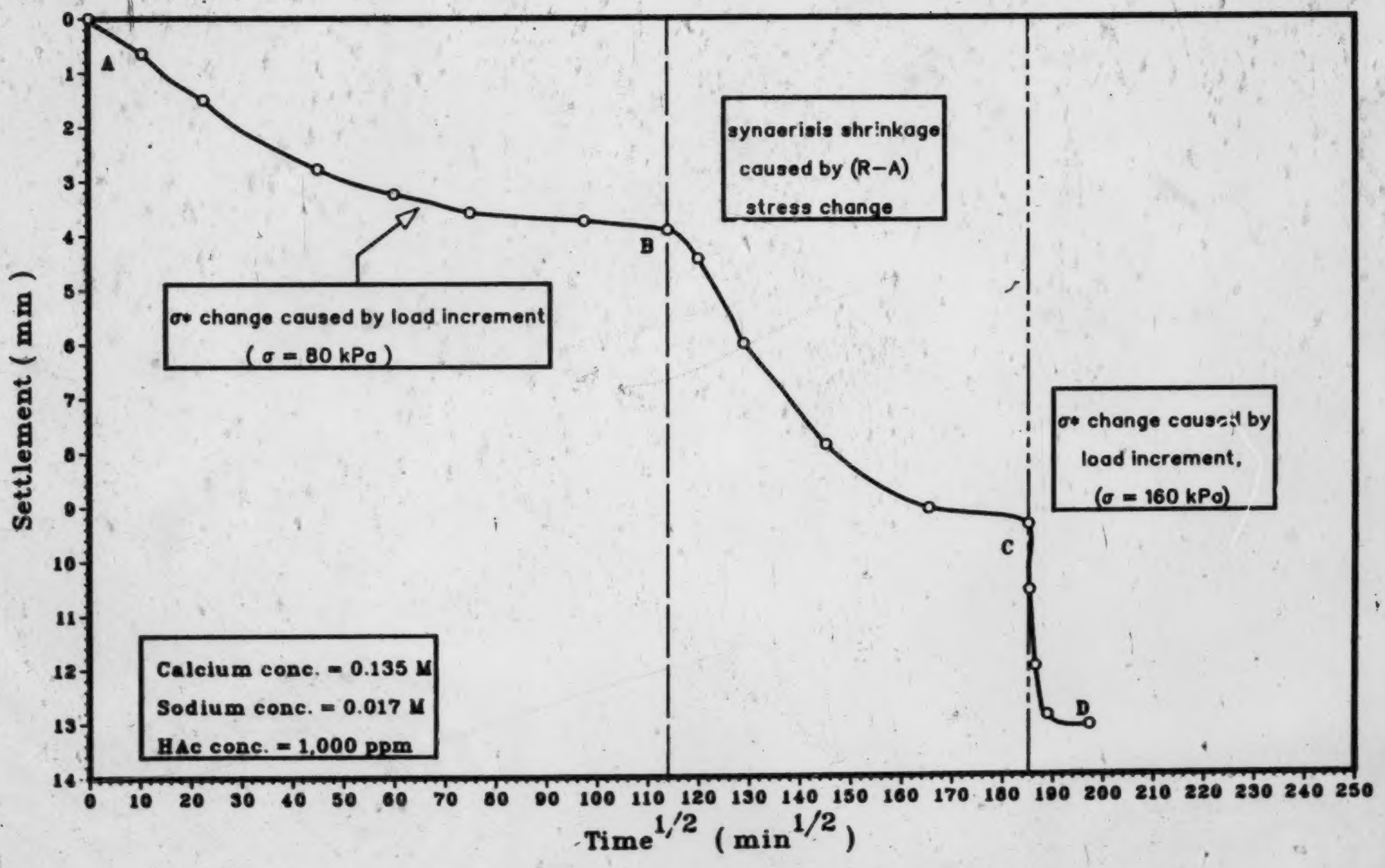

Figure 5.2 Time Settlement Curve for cell C–1.5

The samples contained pure clay minerals, montmorillonite, kaolinite and small amounts of illite. The bentonite used in these samples contained 85% Na montmorillonite, and would swell to a very large volume. Therefore, in essence, the samples contained about 57% montmorillonite and the rest low swelling kaolinite and illite. The major exchangeable cation was Na^+, occupying almost 95% of the exchangeable sites in the montmorillonite.

The high swelling potential of these soils makes them amenable to large volume shrinkages, because of true effective stress, σ^*, increases.

Once the liquid in the leachate reservoir was changed to the test liquid, the electrolyte starts its slow migration through the sample expelling the existing pore water. The concentration of ions within the pore water changes during this process and would reach an equilibrium value after a period of several days. This is evident in Figures 5.1 and 5.2, where the time-dependent, effective stress change, during leachate interaction, created slow settlement rates (as compared with faster settlements observed with external stress increments) and samples needed a longer time to achieve equilibrium with the new environment.

In Figure 5.2, when the test liquid contained high cation concentrations, although the initial settlement rate (given by the slope of the BC portion of the curve) was slower, the settlement progressed at a faster rate after a time lapse of about 20 hours.

Electrolyte migration was controlled both by mass flow and ion diffusion. The rate of settlement held constant at 0.028 mm/h for about 6 days before slowing down. The pore pressures developed, during this effective stress increase, dissipated slowly and it was necessary to monitor the deflection for a period extending 21 days, in order to determine the total volume shrinkage.

The rate of settlement was much lower in the cell C-1.2, (Figure 5.1), where the test liquid contained smaller cation concentrations. Test liquids, in cells C-1.2 and C-1.5, differed mainly in Ca^{+2} concentrations, C-1.2 being the cell with lower Ca^{+2} concentration.

Although both contained the same soil, the magnitude of synaerisis shrinkage exhibited by cell C-1.2 was much lower than that by cell C-1.5. The cell C-1.5 charged with Ca^{+2} = 0.135 M, showed a synaerisis consolidation settlement of 5.34 mm (percent strain = 21.50), as opposed to 1.62 mm (percent strain = 5.68) for cell C-1.2, charged with the low Ca^{+2} concentration of 0.009 M.

As evident from the results presented in Table 5.2, leachates containing higher concentrations of Ca^{+2} cause larger shrinkages (greater than 21.0 percent strains). The acetic acid and Na^{+} at higher concentrations showed a lesser effect.

The loss of some of the data points prevented a complete statistical analysis of the series #1 results. One of the main objectives in performing this test run was to evaluate the main effects and the interaction effects. Still, it was possible, at least, to evaluate the main effect of Na^+, Ca^{+2}, H^+ by the following procedure (Davies 1979).

The main effect of a factor is given by the difference between the average response at the higher level (+1) and the lower level (-1). Therefore using the response percent strain values tabulated in Table 5.2, it is possible to calculate the main effects as follows:

$$\text{Main effect of a factor} = \{\text{Average at level } (+1)\} - \{\text{Average at level } (-1)\}$$

$$\text{main effect of } Na^+ = \frac{15.50 + 14.80 + 21.67 + 21.48}{4} - \frac{21.70 + 18.81 + 5.68}{3}$$

$$= 2.96 \text{ units.}$$

$$\text{main effect of } Ca^{+2} = \frac{21.70 + 21.67 + 21.48}{3} - \frac{14.80 + 15.50 + 18.81 + 5.68}{4}$$

$$= 7.92 \text{ units.}$$

$$\text{main effect of HAc} = \frac{21.48 + 14.80 + 18.81}{3} - \frac{21.70 + 21.67 + 15.50 + 5.68}{4}$$

$$= 2.22 \text{ units.}$$

These values show that the effect of $Ca^{+2} >> Na^+ >$ HAc. However, in view of the smaller effect shown by HAc in general, percent strain values obtained for cell C1.3 seems to be higher than expected.

Cell C-1.3 was charged with 20,000 mg/l acetic acid and low concentrations of Na^+ and Ca^{+2}. The recorded percent strain was 18.81. Compared to C-1.3, cell C-1.7, charged with high Na^+ and HAc concentrations (and low Ca^{+2}), recorded a lower 14.80 percent strain. This is not expected if the main effects are additive. Furthermore, preliminary sedimentation test results indicated a larger percent strain for cell C-1.7 (shown in Figure 5.13) rather than for cell C-1.3. This observation could be explained in two ways. First, higher shrinkage value for cell C-1.3 may have been caused by an experimental error. Therefore, this trial was repeated in a later series (series #7). The percent strain obtained was 16.02 (the void ratio, log pressure summary is provided in Appendix F). The percent strain in the repeat was 14% lower than the value reported in series #1 and was well above the variance reported for this set of experiments. The standard deviation calculated for the repeat cells C-1.5, C-1.10 and C-1.12 was only 2.2%. However, the sensitivity analysis (Section 4.3.2.3) for the consolidation results indicated the low reproducibility at very high bentonite contents. Therefore, it would seem worthwhile to look at another explanation for the higher-than-expected percent strain values for cells C-1.3 and the repeat.

The sediment volume results obtained from sedimentation experiments could only indicate the volume changes expected from a change in interparticle, physico-chemical forces or the double layer thicknesses. But with high acetic acids, mechanisms other than physico-chemical forces could contribute to the observed shrinkage. Acetic acid and the very low pH environment (pH ≃ 2.3, for this liquid combination) could dissolve the aluminosilicates in the clay mineral. Although an ongoing reaction was observed in sedimentation tubes (tiny bubbles formed within the soil-electrolyte after several days) its effect was not reflected in the sedimented volume. However, such a dissolution reaction could contribute to the breakage of clay mineral structure and the collapse of the soil fabric, resulting in a larger volume shrinkage than expected from only the interparticle physico-chemical force considerations.

Three cells contained the upper level acetic acid concentration (cells C-1.3, C-1.7 and C-1.9). Cell C-1.9 contained upper levels for all three factors and a major contribution could be due to synaerisis, with the effect of acetic acid diminished by the presence of high Ca^{+2} and Na^{+}. In cell C-1.7, presence of Na^{+} itself does not contribute greatly to synaerisis (as compared to Ca^{+2}), and exhibited a lower shrinkage than cell C-1.3. However, further experiments were needed to evaluate and understand the acetic acid effect. The explanation given here was not discounted at this stage of the research program.

After charging the cells with leachates and leaving them for several days, a major problem seem to arise with some of the cells. In the high salt environment, the steel balls used for load transmission to the soil sample showed signs of corrosion taking place. Corrosion was severe in cell C-1.6, and affected the final shrinkage values (as evident in Figure 5.3). Unlike the other cells, in which the shrinkage reduced to a minimum (settlement levelling off) after about 3 weeks, shrinkage did not diminish with time (after a period of time). With the addition of more Fe^{+2} ions to the solution by corrosion, shrinkage kept on. At this point in the research program, it was decided to replace the steel balls (which were found to be nickel plated high carbon iron balls) by teflon balls of the same size.

Experimental Series #2

Series #2 experiments were performed with soil samples containing a smaller bentonite percentage. Again, a 2 level factorial design was employed to study the effect of cation Na^{+}, Ca^{+2} and HAc. The design matrix and the levels of each factor are tabulated in Table 5.3. The synaerisis shrinkage results for each consolidation cell is given in Table 5.4. The void ratio-log pressure results, calculated for each cell are included in appendix F as Tables F2.1 - F2.7. All nine time settlement curves are included in appendix E (Figures E2.1 to E2.4).

In fact, series #2 experiments were conducted to confirm the outcome of series #1 and to determine if the

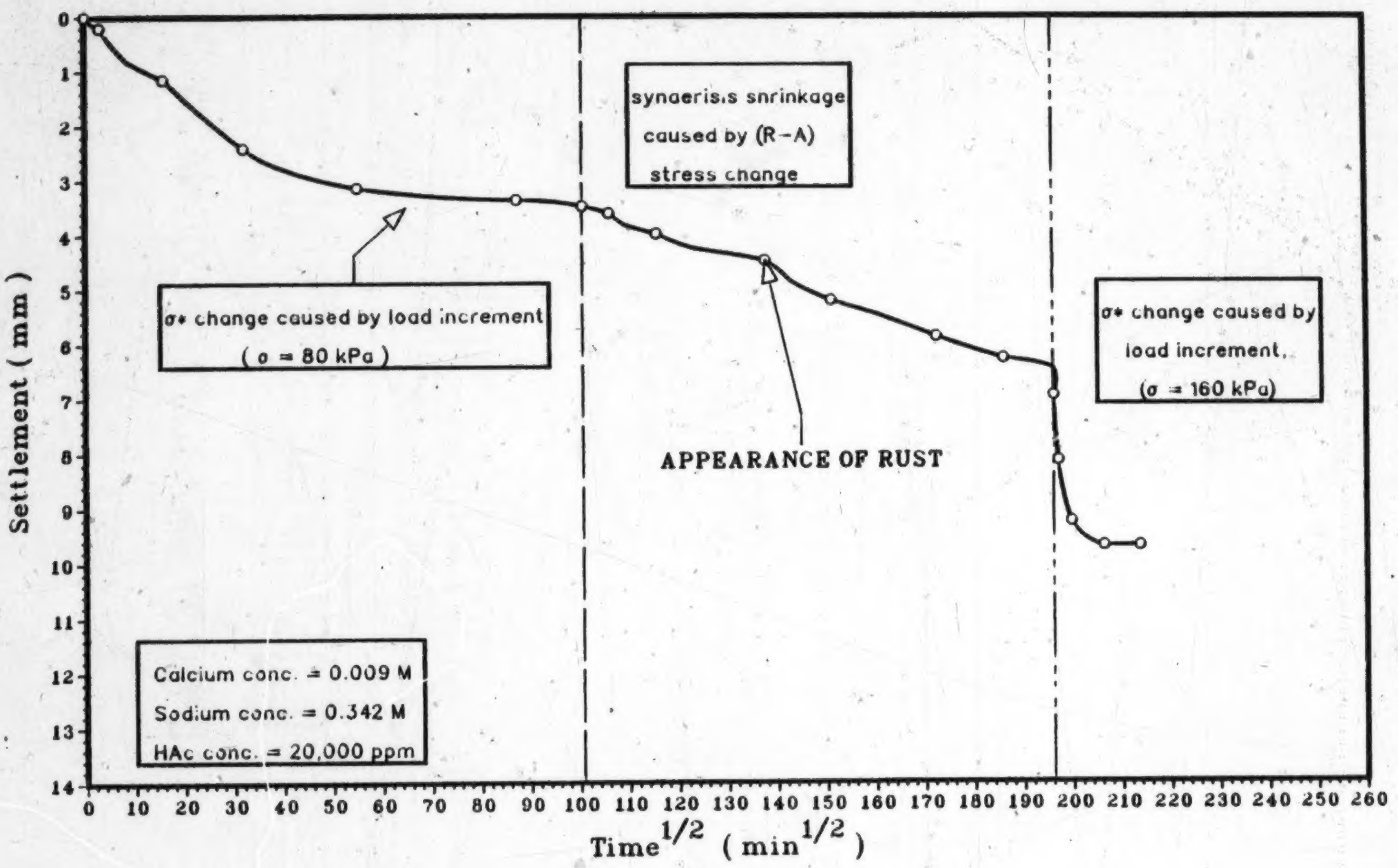

Figure 5.3 Time Settlement Curve for Cell C-1.6

TABLE 5.3

SERIES #2 EXPERIMENTAL DESIGN

---

a) DESIGN MATRIX

Similar to Series #1 and Table 5.1 (a)

b) FACTORS AND LEVELS

Same as Series #1 and Table 5.1 (b)

c) SOIL

Bentonite = 20 % (i.e. Na mont. = 17 %
and Illite = 3 %)

Kaolinite = 80 %

TABLE 5.4

SUMMARY OF SYNAERISIS CONSOLIDATION RESULTS - SERIES #2

CELL #	SOIL TYPE	DRY SOIL WEIGHT (grams)	AMOUNT Na MONT. (grams)	INITIAL HEIGHT (Before Leachate Addition) (mm)	Na	Ca	HAc	TOTAL DEFLECTION (Due to Leachate Addition) (mm)	PERCENT STRAIN (%)
C 2.1		76.94	13.08	30.40		CONTROL (DEIONIZED WATER)			0.00
C 2.2		76.73	13.04	27.80	-1	-1	-1	0.32	1.17
C 2.3		76.97	13.08	27.26	-1	-1	+1	1.37	5.01
C 2.4		76.56	13.02	27.74	-1	+1	+1	2.04	7.36
C 2.5		76.70	13.04	30.80	-1	+1	-1	1.92	6.22
C 2.6	Bentonite = 20 %	76.63	13.03	28.52	+1	-1	-1	1.38	4.83
C 2.7	Kaolinite = 80 %	76.80	13.06	29.30	+1	-1	+1	1.49	5.08
C 2.8	(passing	76.77	13.05	30.40	+1	+1	-1	1.70	5.58
C 2.9	#50 sieve)	76.42	12.99	28.42	+1	+1	+1	1.87	6.58
C 2.10		(EXPERIMENT		FAILED)	0	0	0	—	—
C 2.11		(EXPERIMENT		FAILED)	0	0	0	—	—
C 2.12		(EXPERIMENT		FAILED)	0	0	0	—	—

different clay mineral percentages had any effect on the significance of the main factors. It was necessary to verify some of the findings, for example, the anomalous behavior of cell C-1.3 in a similar experiment, under different conditions. In this run, 3 cells had to be abandoned because of the problems that arose during sample preparation and leachate permeation stages. Tilting of the loading cap, corrosion of metallic parts and growth of a microbial population were the chief problems.

The design matrix and the levels for each factor were tabulated with the response (% strain) for each consolidation cell (see Table 5.4). The main effect of each factor was calculated and compared with each other using the method discussed earlier. In this case, the values obtained again showed the high significance of the Ca^{+2} effect with: Ca^{+2} >> HAc >> Na^{+}. The corresponding values were 2.41, 1.56 and 0.58 for Ca^{+2}, HAc and Na^{+}, respectively.

The main factors and interactions were evaluated according to the Yates' method of analysis of variance (ANOVA) as summarized in Davies (1979) on Table 5.4 data. The mean square (MS) of each term was calculated from the total effect (TE), which in turn was calculated using the regression coefficients (β) obtained by performing a multivariate least squares regression of the design matrix on the response, percent strain.

The error mean square (MSE) may be calculated from centre point replicates or from the mean square of an insignificant main effect or an interaction. Since the centre point replicate results were lost, and the significance of each of the main effects or interactions could not be verified beforehand, it was decided to use the MSE obtained for the low % bentonite replicates in the sensitivity analysis, results of which are presented in Section 4.3.2.3.

The ratio of MS/MSE produced a statistic which was compared to the corresponding F statistic. Those factors and interactions exceeding the tabulated F-statistic at 0.05 level, had statistically significant effects on the response at the reference level. The ANOVA table for series #2 is presented in Table 5.5.

The ANOVA results are consistent with the double layer theory predictions. The divalent cation, Ca^{+2}, plays the major role in volume shrinkage, as expected. Replacement of Na^{+} ions in the exchangeable positions by divalent Ca^{+2} ions caused a reduction in long range repulsive stress, R_1. The magnitude of change is larger in C-1 samples (series #1) mainly because those samples contained higher percentages of Na montmorillonite.

The highly significant influence of Ca^{+2} concentration, and the marginal influence of Na^{+} and HAc concentration can be explained by considering the cation exchange-equilibrium and the resulting interparticle stress changes within the soil system. The following discussion focuses on the extent of exchange complex domination by the divalent cation. The

TABLE 5.5 RESULTS FROM ANALYSIS OF VARIANCE

(SERIES # 2)

FACTOR	REGRESSION COEFFECIENT (B)	MEAN SQUARE (MS)	ERROR MEAN SQUARE (MSE)	MS/MSE	SIFNIFICANCE (at 95%)
Na	0.289	0.334	0.054	6.22	NOT SIGNIFICANT
Ca	1.204	5.796	0.054	108.12	HIGHLY
HAc	0.791	2.504	0.054	46.73	SIGNIFICANT
Na.Ca	-0.634	1.607	0.054	29.97	SIGNIFICANT
Na.HAc	-0.231	2.133	0.054	39.79	SIGNIFICANT
Ca.HAc	-0.730	0.214	0.054	3.99	NOT SIGNIFICANT

effect this would have on the interparticle stress system will be discussed extensively in section 6.2.

In heterovalent systems, theoretical evidence (discussed in Chapter 3) points to a domination of the exchange complex by divalent cations. For example, if a Gapon K_G value of 0.017 (Mitchell, 1976) was assumed for the clay soil, which was equilibrated with the leachate solution containing 500 meq/L $CaCl_2$ and 435 meq/L NaCl, the ratio of exchangeable sodium to calcium would be,

$$\frac{Na_e}{Ca_e} = 0.017 \times \frac{435}{\sqrt{250}} = 0.47$$

This would cause an exchange of most of the Na^+ ions in the Na montmorillonite to Ca^{+2} ions and a resulting double layer thickness diminution. The series #1 and #2 results provide the experimental evidence.

Experimental Series #3

Series #3 experiments were performed to explore and expand further the experimental region of the most significant factor, Ca^{+2} concentration.

A single factor, multiple level experimental series was designed for this purpose. The "one factor at a time" design provides an estimate of the effect of a single variable at selected, fixed conditions of the other variables. This method was suitable because the interactions due to other variables were minimal (or maximizing value of Ca^{+2} is independent of others).

The montmorillonite content in Alberta soils varies from a low 20% in Northern Alberta to a high 60% in South-Central Alberta (Appendix B2). Therefore, 40% bentonite (34% Na montmorillonite) was selected for this series, corresponding to the average montmorillonite content in Alberta soils.

The pH in leachates ranged from 3.7 to 8.5.(Table 3.2) Therefore, the use of large amounts of acetic acid, which invariably reduces the pH to below 3.7, is unjustified. H^+ ions join Na^+ and Ca^{+2} cations in affecting the interparticle forces and a clear understanding of the physico-chemical basis of the interaction may not be feasible. Therefore, acetic acid concentration was maintained at a low 250 mg/L for series #3 experiments. NaCl concentration was maintained at 1000 mg/L (corresponding Na^+ conc. = 0.017 moles/L).

Time settlement curves for all 9 cells were plotted and presented in Appendix E as Figures E3.1 to E3.9. Void ratio log pressure results are presented in Appendix F, as Tables F3.1 to F3.9.

Use of teflon balls and soil passing #200 sieve allowed the experiments to be performed in a trouble free environment. The synaerisis shrinkage results thus obtained are presented in Table 5.6, together with the other relevant information. The initial void ratios for the 9 cells showed an average value of 3.59 and a 8% standard deviation, which

TABLE 5.8

SUMMARY OF SYNAERISIS CONSOLIDATION RESULTS - SERIES #3

CELL #	SOIL TYPE	DRY SOIL WEIGHT (grams)	AMOUNT Na MONT. (grams)	INITIAL HEIGHT (Before Leachate Addition) (mm)	INITIAL VOID RATIO (Before Leachate Addition)	LIQUID Ca Conc. (moles/1)	TOTAL DEFLECTION (Due to Leachate Addition) (mm)	PERCENT STRAIN (%)
C 3.1	Bentonite=40% Kaolinite=60% (passing #200 sieve)	59.51	20.23	28.55		CONTROL		0
C 3.2		59.45	20.21	27.53	3.71	BASE ONLY	0.10	0.36
C 3.3		59.56	20.25	26.51	3.53	0.002 + BASE	0.25	0.94
C 3.4		59.42	20.20	26.48	3.74	0.004 + BASE	0.24	0.91
C 3.5		59.56	20.25	26.87	3.58	0.009 + BASE	0.69	2.57
C 3.6		59.45	20.21	26.90	3.67	0.016 + BASE	1.88	6.99
C 3.7		59.31	20.16	26.35	3.47	0.022 + BASE	2.79	10.59
C 3.8		59.45	20.21	26.64	3.54	0.045 + BASE	3.53	13.25
C 3.9		59.45	20.21	26.69	3.51	0.090 + BASE	3.99	14.95

Base liquid consists of :

- 0.017 moles/1, Na
- 250 mg/1, HAc

indicated the good reproducibility of the sample preparation technique. When comparing effect of various calcium concentrations on the shrinkage behavior, it is essential to start with soil samples exhibiting similar characterisitics. The liquids produced comparativley small percent strains, above control value, at lower Ca^{+2} concentrations. The "base" liquid alone (containing 0.017 moles/L Na^{+} and 250 mg/L HAc) produced only 0.1 mm (0.36 percent strain) shrinkage.

The percent strain caused by synaerisis was plotted against the calcium concentration in Figure 5.4. Almost a linear increase of synaerisis shrinkage was observed between Ca^{+2} concentrations 0.005M and 0.02M. Difficulty in obtaining good samples prevented analysis of porewater for each of the cells before leachate permeation. However, from the analysis of several samples, an average value for the Na^{+} concentration at the beginning of leachate permeation was determined as 0.004 M. The addition of base liquid would not increase the electrolyte concentration to levels capable of reducing double layer thicknesses. However, addition of higher Ca^{+2} concentrations displaced the Na^{+} from the clay micelle and reduced the bound water, thereby changing the interparticle forces. This would continue until the majority of the Na^{+} ions are removed from the particle exchange spaces (the straight line portion in Figure 5.4). Once the Na^{+} ions are removed from the vicinity of the clay particles, higher Ca^{+2} concentrations would not affect the

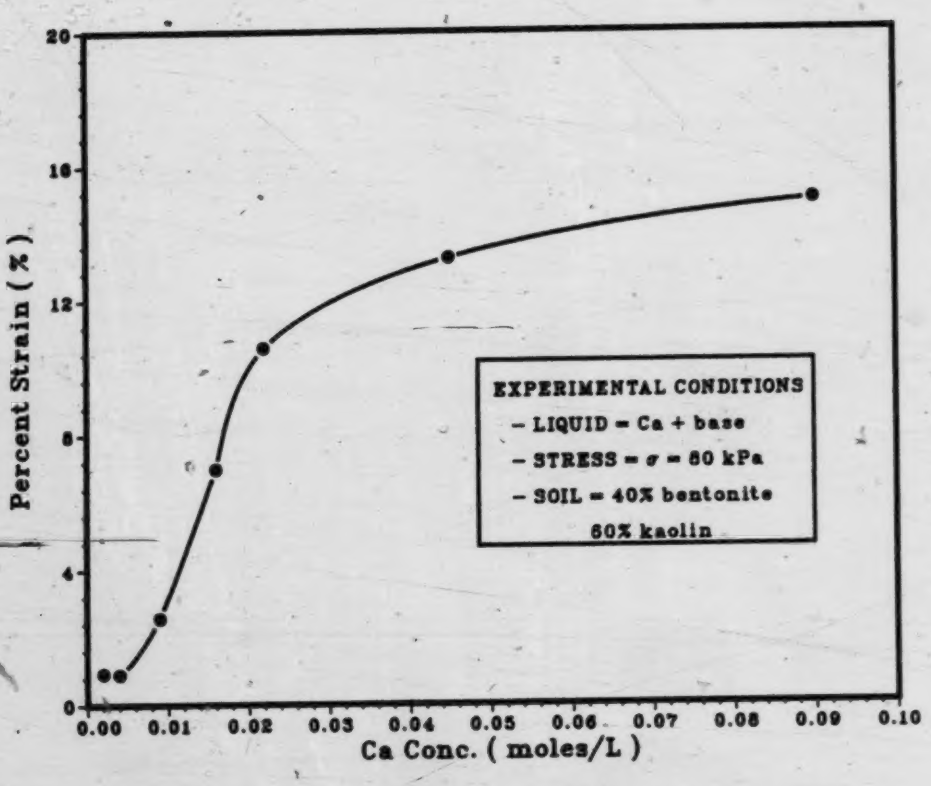

Figure 5.4 Percent Strain Vs Calcium Concentration

characteristics of the bound water and the resulting interparticle forces (further explanation of the role interparticle forces play in controlling the synaerisis phenomenon is given in the next Chapter). Therefore, after the rapid increase, the percent strains would not change to a considerable degree with the increasing Ca^{+2} concentrations. The hypothetical extension of the plot between % strain and Ca^{+2} concentration would indicate a maximum percent strain of 16% for this soil (containing 40% bentonite and 60% kaolinite). It should be noted that 84% of this strain could be achieved with only 0.045 M Ca^{+2} (5,000 mg/L $CaCl_2$ plus base).

5.2.2 Effect of Clay Mineral Type and Exchangeable Cations

To perform the first three series of experiments, kaolinite was mixed with bentonites. Results indicated that the kaolinites contributed very little to the physico-chemical behavior of these test samples. Thus, samples containing higher percentages of bentonite consistently showed higher volume shrinkage.

Results obtained in series #1 and #2 are directly comparable, in terms of their shrinkage behavior with respect to the percentage bentonite content. Figure 5.5, exhibits the effect of % bentonite on the percent synaerisis strain. Seven liquid combinations are compared. In each case, cells containing 67% bentonite produced 3 to 4 times higher shrinkages than the cells containing 20% bentonite.

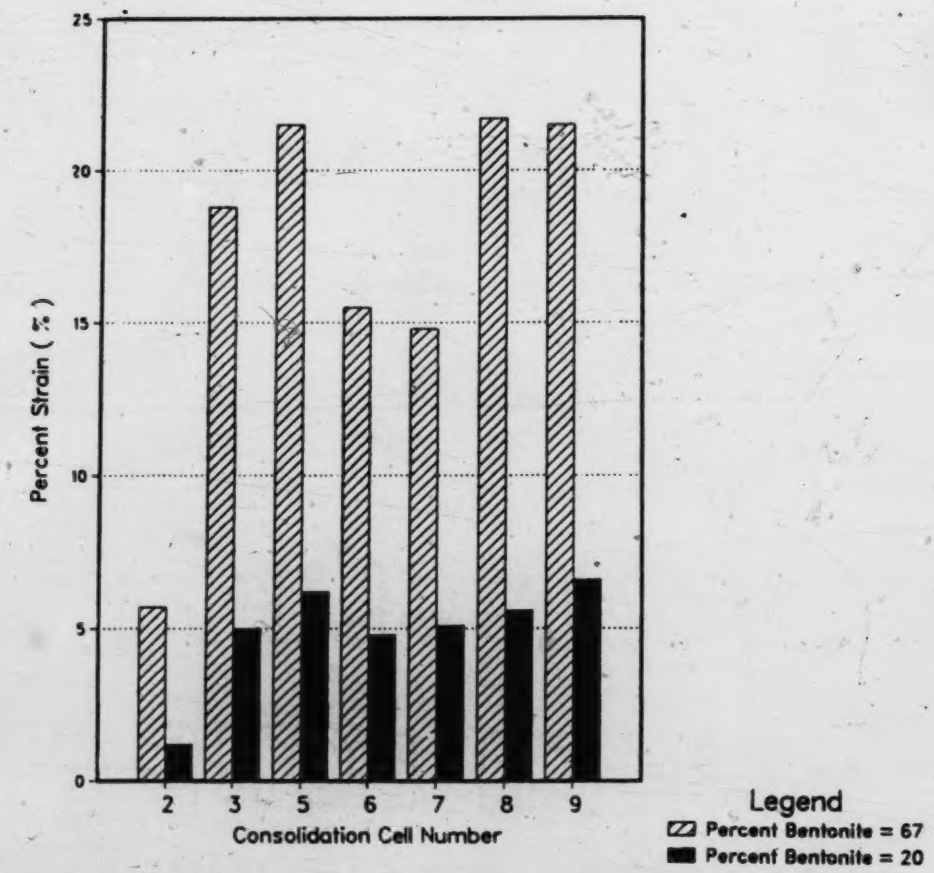

Figure 5.5 Effect of % Bentonite on Synaerisis Shrinkage

By considering series #1 and #2 experiments as two blocks of a larger randomized block design, it can be easily shown that a highly significant difference exists between these two blocks.

By studying the series #1 and series #2 results, an important feature of the soil shrinkage phenomenon still could not be fully assessed. The magnitude of shrinkage clearly depends upon the percentage of bentonite, showing almost a linear relationship (see Table 5.7). The series #1 cells, which showed higher shrinkage values, contained higher amounts of bentonite. However, the increase in shrinkage was not in direct proportion to the increase in amount of bentonites. Further experiments were needed to clarify this point.

The series #5 experiments were performed to help establish the following:

- importance of percentage Na montmorillonites in governing the synaerisis shrinkage behavior of a soil,
- role of the exchangeable cation in controlling the development of interparticle stresses within a clay water system.

Therefore, it was necessary to use clay samples containing the following clay minerals:

- kaolinite and Na montmorillonite (from bentonite),
- Na montmorillonite and Ca montmorillonite.

TABLE 5.7

RELATIVE INFLUENCE OF PERCENT BENTONITE AND AMOUNT BENTONITE ON PERCENT STRAIN (FROM SERIES #1 & 2)

LIQUID COMBINATIONS (from Table 5.2 & 5.4)	PERCENT STRAIN	
	20% Bentonite (C2 Cells)	67% Bentonite (C1 Cells)
#2 (-1 -1 -1)	1.17 (13.04)	5.68 (18.34)
#3 (-1 -1 +1)	5.01 (13.08)	18.81 (18.31)
#4 (-1 +1 +1)	7.36 (13.02)	—
#5 (-1 +1 -1)	6.22 (13.04)	21.50 (18.06)
#6 (+1 -1 -1)	4.83 (13.03)	15.50 (18.46)
#7 (+1 -1 +1)	5.08 (13.06)	14.80 (17.93)
#8 (+1 +1 -1)	5.58 (13.05)	21.67 (18.18)
#9 (+1 +1 -1)	6.58 (12.99)	21.48 (18.13)

NOTE :

Corresponding grams of Bentonite are given within brackets.

The dominant cation in natural clay soils is Ca^{+2}. This seems to be true with Alberta soil, too. Therefore, it was appropriate to test the pure calcium montmorillonite, along with the other two clay minerals, kaolinite and sodium montmorillonite.

In preliminary studies, Ca montmorillonite did not swell to large volumes in deionized water, unlike the Na version. The FSDP values (to be discussed in Section 5.3), obtained by sedimenting the soil in test liquids (containing various amounts of Ca^{+2}, Na^{+} and acetic acid), were extremely low. In fact, the sedimentation experiments revealed that volume change behavior of Ca montmorillonite closely resembles that of kaolinite.

Therefore, the swelling pressure in a prepared sample (by consolidating up to a pressure of 80 kPa) is low, and the magnitude of shrinkage expected when permeated with high electrolyte leachate would be small. The swelling/shrinking behavior of Ca montmorillonite would be closer to the kaolinite than to Na montmorillonite.

Based on the first 3 experimental runs, and the average content of electrolytes in the leachate, it was decided to use a standard leachate for series #5 experiments. (NOTE: Series #4 experiments were performed specifically to determine the sensitivity of the synaerisis consolidation test and results were included in Chapter 4). This standard leachate constituted of :

- 0.045M calcium ions (5,000 mg/L of $CaCl_2$),

- 0.017M sodium ions (1,000 mg/L of NaCl),
- 250 mg/L acetic acid (pH = 4.2).

Seven of the twelve consolidation cells were loaded with the clay mineral, montmorillonite; Na^+ and Ca^{+2} were mixed at varying percentages. The results obtained by performing synaerisis consolidation tests on Na, Ca montmorillonite mixtures are reported in Table 5.8.

The cells containing higher percentages of Na montmorillonite showed larger percent strains. The dependency of the magnitude of synaerisis shrinkage on the percent bentonite content is clearly seen in Figure 5.6. However, it was important to see whether such a relationship was available between amount of bentonite and percent strain. This is not evident in Figure 5.7. Cell C-5.3 and C-5.4 contained Na montmorillonite, equal to 17.14 and 17.78 grams, respectively. Based on the amount of Na montmorillonite, it was not possbile to predict the observed shrinkage differences between these two cells. Cell C-5.3 (% bentonite = 55) showed a 13.82 percent strain, whereas cell C-5.3 containing a higher amount of Na montmorillonite (% bentonite = 40), showed only a percent strain value of 5.15.

In Figure 5.6, the curve corresponding to Ca montmorillonite shows a shallow gradient at low percentages of bentonite. The physico-chemically "inert particles" (kaolin and Ca montmorillonite particles) tend to reduce the double layer development around highly expanding Na montmorillonite particles. By keeping Na montmorillonite

TABLE 5.8

SUMMARY OF SYNAERISIS CONSOLIDATION RESULTS - SERIES #5

CELL #	SOIL TYPE Bentonite (%)	Kaolinite (%)	Calcium Mont. (%)	DRY SOIL WEIGHT (grams)	AMOUNT Na MONT. (grams)	INITIAL HEIGHT (Before Leachate Addition) (mm)	INITIAL VOID RATIO (Before Leachate Addition)	TOTAL DEFLECTION (Due to Leachate Addition) (mm)	PERCENT STRAIN (%)
C 5.1	85	—	15	25.89	18.71	22.80	6.53	6.34	27.81
C 5.2	70	—	30	30.87	18.37	23.61	5.54	4.60	19.48
C 5.3	55	—	45	36.68	17.14	24.38	4.68	3.37	13.82
C 5.4	40	—	60	52.31	17.78	30.06	3.91	1.55	5.15
C 5.5	25	—	75	71.54	15.21	33.86	3.05	0.55	1.62
C 5.6	10	—	90	90.89	9.43	31.02	1.92	0.22	0.71
C 5.7	5	—	95	116.10	5.79	35.59	1.62	0.08	0.22

NOTE : ALL SAMPLES WERE PERMEATED WITH THE "STANDARD LEACHATE"

Ca Conc. = 0.045 M

Na Conc. = 0.017 M

HAc Conc. = 250 mg/1

pH = 4.2

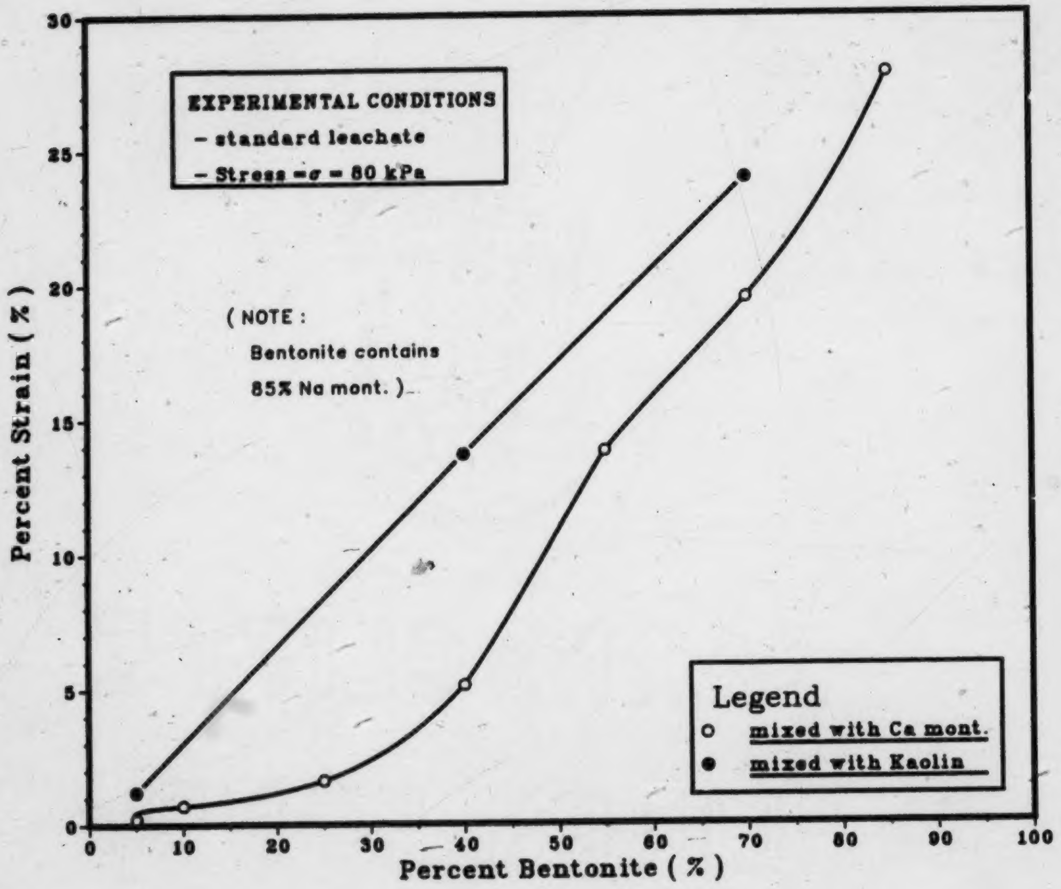

Figure 5.6 Percent Synaerisis Strain Vs Percent Bentonite

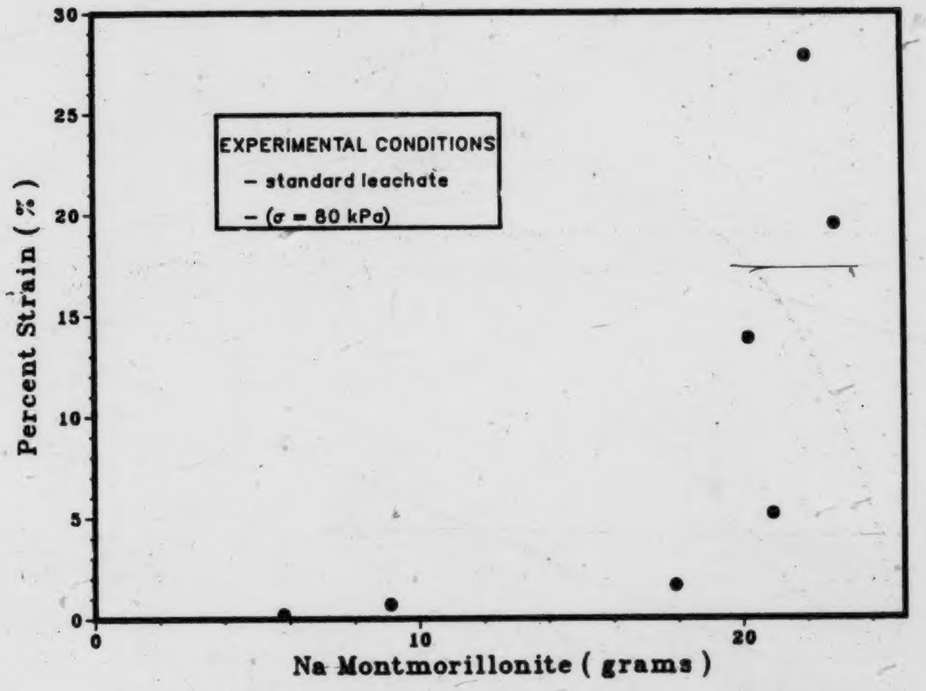

Figure 5.7 Percent Strain Vs Amount of Na Montmorillonite

particles apart by physical interference, kaolinites and Ca montmorillonites are capable of reducing the development of interparticle repulsive stresses. Ca montmorillonites can cause further reductions by injecting Ca^{+2} ions into the pore water.

Experiments C-5.8 and C-5.9 were performed using bentonite:kaolinite 70:30 and 5:95 mixtures, respectively. Results obtained earlier contained a 60:40, bentonite:kaolinite mixture. These results are compared with corresponding bentonite, Ca montmorillonite mixtures in Table 5.9 and in Figure 5.6. The bentonite/Ca montmorillonite samples showed lower percent strains in all 3 situations, reported in Table 5.9. In Figure 5.6, the kaolin/Na montmorillonite mixture showed a steeper constant slope throughout.

Before pouring into the consolidation cells, the clay mineral mixtures were thoroughly mixed with deionized water and allowed to equilibrate for a period of 2 to 3 days. During this period, the soluble salts and some of the exchangeable cations would be released (or exchanged) to the "free water". Therefore, the pore water at the time of leachate addition contains some salinity. The type and amount of cations present, is dependent on the type and amount of soluble salts and exchangeable cations present in the clay mineral mixture. The Ca montmorillonite is capable of injecting more cations to the free water than the kaolinite. The higher the percentage of Ca montmorillonite

TABLE 5.9

A COMPARISON OF PERCENT STRAIN FROM KAOLINITE : BENTONITE AND Ca MONTMORILLONITE : BENTONITE MIXTURES

PERCENT BENTONITE IN THE MIXTURE	PERCENT STRAIN WITH REST OF THE MIXTURE CONSISTING OF	
	Kaolinite	Ca Montmorillonite
5	1.21	0.54
40	13.70	5.51
70	23.92	19.94

NOTE :

Permeating liquid - Standard Leachate

present, the higher would be the amount of Ca^{+2} ions injected to pore water. The higher cation (mainly Ca^{+2}) concentration diminishes the net (R - A) force, and would contribute to lessen the final volume shrinkage caused by the standard leachate. This explains the shallow slope observed, at low bentonite or higher Ca montmorillonite content. This is evident until the % bentonite reaches about 25%. Because of the important implications this would have in modifying problem soils, this idea was pursued further in a subsequent experimental run and is discussed in Section 5.2.5.

Time settlement curves developed for the nine tests are included in Appendix E, as Figures E5.1 to E5.9. A void ratio versus log pressure plot was prepared for cell C-5.8. This plot (Figure 6.4), together with Figures 6.1 and 6.3 are discussed and interpreted, in Chapter 6, with respect to the effective stress concept introduced in Chapter 3. The remaining information relevant to the void ratio vs effective stress plots are tabulated and presented in Appendix F as Tables F5.1 to F5.9. The adopted procedure for this experimental series did not include a 160 kPa stress increment.

5.2.3 Effect of the External Vertical Stress

To this point in the experimental program, the synaerisis consolidation was measured on samples initially constrained at an external stress of 80 kPa. This corresponds to a landfill height of about 10 m. This was considered representative for a landfill in Alberta. However, the landfill heights vary considerably and the corresponding axial stress on a constructed liner varies accordingly.

Although it was not intended to study an expanded experimental region of this variable, series #6 was designed to understand the effect (if any) of the constraining pressure on the synaerisis phenomenon. Since it was designed as a comparative study, sufficient cells needed to be assigned for each axial pressure. Therefore, it was decided to use 6 cells, each to study the behavior at 40 kPa and 160 kPa of mixtures of bentonite and Ca montmorillonite. Employing the same preparation as in series #5 and the standard leachate, it was possible to generate results that are directly comparable to results presented in Table 5.8. The results are compared in Table 5.10. Notwithstanding the variations in the axial stresses applied, the percent strains increased with the increasing % bentonite in the soil samples.

A two-way analysis of variance (ANOVA) was performed, together with a Duncans multiple range test, to ascertain whether a statistically significant difference exists between the three treatments (surcharge pressures). The ANOVA showed F ratio to be insignificant. However, the Duncans multiple range test indicated that the means of the three sets of data are not significantly different from each

TABLE 5.10

PERCENT SYNAERISIS SHRINKAGE STRAINS AT THREE DIFFERENT AXIAL PRESSURE (σ) VALUES (With Standard Leachate)

SOIL MATERIAL		PERCENT STRAIN AT A SURCHARGE PRESSURE OF		
% Bentonite	% Ca Montmorillonite	40 kPa	80 kPa	160 kPa
10	90	0.80	1.08	0.78
25	75	2.31	1.96	1.93
40	60	6.76	5.91	6.29
55	45	13.57	14.28	13.68
70	30	19.51	19.94	20.69
85	15	20.19	28.28	24.65

other. Therefore, based on this analysis, it is possible to say that, at least within the pressures tested, percent strain was not dependent upon the surcharge pressure.

Another factor that would seem to affect the synaerisis shrinkage is the void ratio at the point of leachate addition. Results obtained in series #5 and #6 provided enough information to verify the dependency of synaerisis shrinkage on void ratio.

During the sample preparation stage, each of the 18 soils tested in series #5 and #6 were consolidated to different void ratios. These initial void ratios are shown in Table 5.11. At the same pressure, high montmorillonite content soil consolidated to a larger void ratio. This shows the difficulty in removing the Na montmorillonite associated water (bound water) from the soil water system. In general, the higher pressures were able to remove larger amounts of water and resulted in creating samples with lower void ratios (the voids in the saturated soil were completely filled with water).

Void ratios, thus obtained, were plotted against the respective volume deflections incurred by the leachate interaction. Figure 5.8 shows that the void ratio-synaerisis shrinkage curve is not a unique one. As noted earlier, initial void ratio was a function of both Na montmorillonite content and the axial pressure.

Higher void ratios are observed both at low surcharge pressures and high Na montmorillonite ratios. Therefore, the

TABLE 5.11

VOID RATIO BEFORE LEACHATE ADDITION
(Similar Soil Mixtures at Different Consolidating Pressures)

SOIL MATERIAL		VOID RATIO AT A CONSOLIDATION PRESSURE OF		
% Bentonite	% Ca Montmorillonite	40 kPa Cells C 6-1 to C 6-6	80 kPa Cells C 5-1 to C 5-6	160 kPa Cells C 6-7 to C 6-12
85	15	8.39	6.59	4.68
70	30	6.68	5.61	3.97
55	45	5.16	4.73	3.25
40	60	3.92	3.94	2.61
25	75	3.12	3.06	2.31
10	90	2.60	1.82	2.12

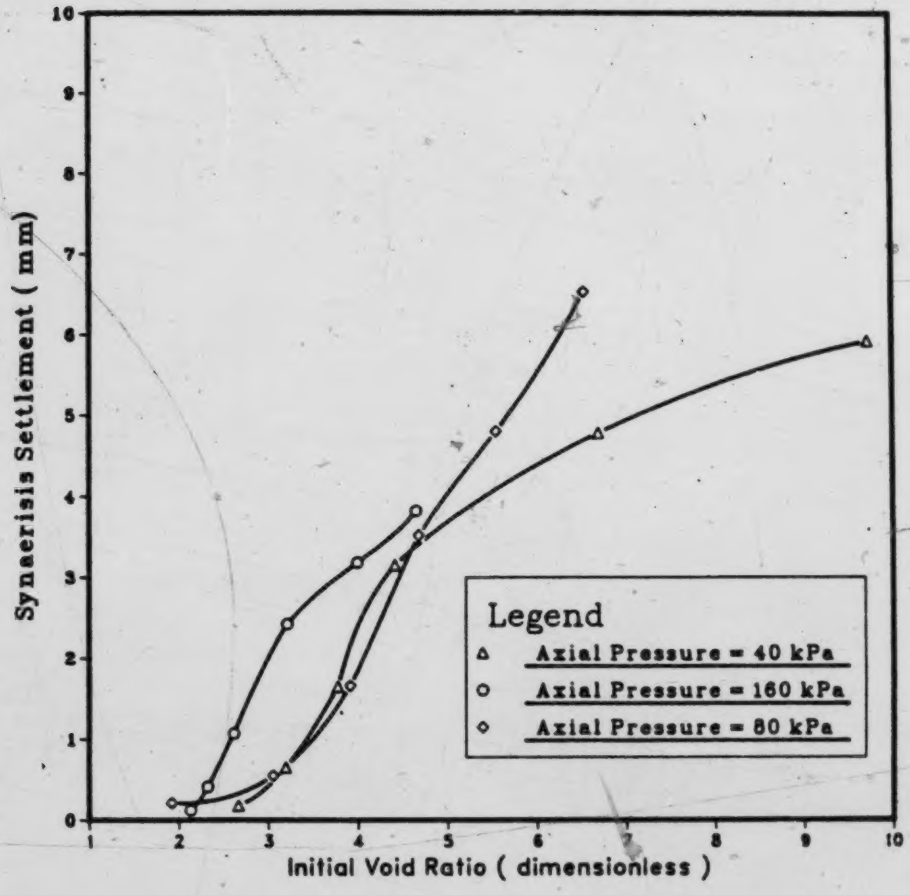

Figure 5.8. Synaerisis Settlement vs Initial Void Ratio

high shrinkages achieved are attributed to the high Na montmorillonite content rather than higher void ratio. This can be clearly shown with specific examples. Cell C-6.8 contained 70% bentonite and was consolidated by a pressure of 160 kPa to a void ratio of 3.97. Cell C-5.4, containing 40% bentonite, was also consolidated to a comparable void ratio of 3.94 by adopting the lesser 40 kPa pressure. Cell C-6.8 exhibited a shrinkage of 3.28 mm (20.69 percent strain), whereas for cell C-5.4 deflection was only 1.77 mm (6.76 percent strain), under standard leachate charging. Similarly, C-6.6 (10% bentonite at 40 kPa and a void ratio of 2.60) and C-6.10 (40% bentonite at 160 kPa and a void ratio of 2.61) gave 0.29 mm or a percent strain of 0.80, and 1.177 mm or a percent strain of 6.29, respectively. These results clearly indicate that, at least within the pressure range of the experiments (40 kPa to 160 kPa), the synaerisis shrinkage is independent of the sample initial void ratio. However, further verifications would be necessary if these findings are to be extrapolated to lower void ratios and higher pressures.

Performance of experiments at very low void ratios was considered necessary when the landfill situation is to be duplicated in the laboratory. Once compacted into a clay liner, the field clay soil would exhibit a void ratio value of about 1.0. This value, depends upon the amount of clay sized particles present in the soil.

To discern the synaerisis effect at very low void ratios, two soil samples were consolidated to a high axial pressure (560 kpa). The reservoir water was changed to the standard leachate, and the deflection was monitored. Total shrinkage was smaller than at 80 kPa surcharge pressure. These results are discussed in detail in section 6.5.2.

5.2.4 Experiments With Natural Soils

The sedimentation volumes in deionized water revealed the non-expansive nature of the three native soils. As will be shown later, an expanding soil would show its largest volume at very small electrolyte concentrations and would show progressively lower volumes with increasing cation concentrations. Howver, with all three soils, no discernible effect of cations was observed (any visible volume change was within the experimental error reported in Chapter 4) with increasing cation concentrations. Subsequent mineralogical and exchangeable cation analysis provided additional evidence in this context.

As reported in Appendix B2, Alberta soils contain varying degrees of highly expanding montmorillonite clay mineral. Clay content could also vary from a low 10-20% in Northeastern Alberta to greater than 70% in Northwestern Alberta. Unfortunately, the two soils provided by Alberta Environment (selected from 8 samples, all exhibiting similar swelling characteristics) and the one obtained from the Geotechnical branch of the Civil Engineering Department did not contain a high clay fraction, and the montmorillonite content only varied between 10% to 20%. In addition, 75% of the exchangeable cations in soils A and B and 65% for soil C, were Ca^{+2}.

Based on the experience with pure clay minerals, a high volume change was not expected with the three natural soils. However, if the volume changes observed in synaerisis consolidation were due to some cause other than changes in interparticle forces, these experiments should be able to resolve such an issue. As seen from Table 5.12, passage of leachate had very little effect on the natural soils.

Measured deflections, without control correction, are 0.095 mm, 0.157 mm and 0.136 mm for soils A, B and C respectively. It was not possible to run 3 controls for these three soils. However, series #2 (20% bentonite) provided an approximate control value (experiments being performed at similar conditions) of 0.108 mm. This value indicated that the deflections obtained for soils A, B, and C to be negligible (taking into consideration the experimental error). All three soils produced a similar response to leachate treatment.

Three other cells, loaded with modified soil mixtures (A, B, and C modified with 10% bentonite) produced some shrinkage when exposed to standard leachate. Values corrected for the "control" are, 0.078, 0.37 and 0.174 respectively. It is worthwhile looking at the reason behind observed differences in the volume shrinkages for soil B and

TABLE 5.12

SYNAERISIS SHRINKAGE RESULTS FOR NATURAL SOILS

CELL #	SOIL	PERCENT STRAIN
7.1	A	0.36
7.3	B	0.59
7.2	C	0.49
7.4	A + 10% bentonite	0.72
7.6	B + 10% bentonite	1.74
7.5	C + 10% bentonite	1.16

soil C (soil C showed only 50% of soil B shrinkage).

Soil B and C were similar in mineralogical composition, containing higher montmorillonite percentage than A. However, as evident from the soluble salt data, soil C is capable of injecting large numbers of both Na^{+} (14.35 meq/L) and Ca^{+2} (22.7 meq/L) into the pore spaces. In comparison soluble salts in soil B were very low (Na^{+} = 2.14 meq/L and Ca^{+2} = 1.3 meq/L). The lower cation concentration in "modified soil B", favoured a development of a higher repulsive stress and subsequent larger changes in the true effective stress (σ^{*}). On the other hand, in modified soil C, higher cation concentration, particularly the presence of large amounts of Ca^{+2} in the pore water, prevented development of double layers around Na montmorillonite particles and diminished the chances of higher repulsive stress development. This essentially reduced the volume shrinkage caused by synaerisis action.

Note also the low initial void ratios exhibited by natural soils (little more than 1.00). The modification of soil with bentonite tended to increase the void ratio, and reduce the compressibility of the soil. As noted with pure clay minerals, soils containing higher percentages of expandable clay minerals (Na montmorillonite) are difficult to consolidate to smaller void ratios. Higher pressures, or soil stabilization by decreasing the bound water (discussed in the next section), are needed to improve the soil compressibility. Time settlement curves are drawn for these

6 soils and included in Appendix E (Figures E7.1 to E7.6). These curves are also used for hydraulic conductivity determinations. Calculated void ratio versus pressure tables, which essentially are summaries of each synaeresis consolidation test (consolidation from a slurry to 160 kPa), are presented in Appendix F (Tables F7.1 to F7.6).

5.2.5 Effect of Soil Treatment and Time Dependent Changes in Leachate Characteristics

One of the main objectives of this study was to propose strategies to eliminate occurrence of excessive volume shrinkages within a MSW sanitary landfill clay liner. The earlier experiments were performed to identify causes and important contributors to the synaerisis phenomenon, which was identified in Chapter 3 as a potential mechanism for shrinkage. The foregoing discussion of experimental results showed that leachate constituents, mainly the divalent cations Ca^{+2}, could cause large shrinkages in swelling soils. Ca^{+2} ions tend to reduce the interparticle repulsive force possessed by the swelling clay soils. As would be proven in Section 6.2, the reduction in the (R-A) stress causes an increase in the true effective stress (σ^*) and results in shrinkage. The net change in (R-A) stress is clearly a function of initial swelling pressure, or a soil treated to remove the development of pre-leachate repulsive stress would not be subjected to substantial volume shrinkage, due to synaerisis.

This concept was tested during experimental series #8. 3 soils, each containing 25% bentonite, were mixed with electrolyte solutions containing 555 mg/L, 1,000 mg/L and 5,000 mg/L $CaCl_2$, instead of the usual deionized water. The resulting slurries were consolidated following the usual procedure and were subsequently charged with the standard leachate.

Higher Ca^{+2} (divalent) concentrations in the pore water are known to reduce the initial long range repulsive stress, R_1. Furthermore, it was revealed in Chapter 3, that most of the Ca^{+2}, ions situated within the clay micelle are concentrated in the Stern layer, closer to the particle. Therefore, it would seem possible, that even moderate amounts (the amount needed to satisfy the cation exchange capacity) would be able to suppress the development of the double layer and minimize the magnitude of R_1.

The synaerisis shrinkage results thus obtained are presented in Table 5.13. The treated soils, when consolidated to 80 kPa, exhibit low swelling pressures, and the subsequent volume reduction, due to synaerisis action, was smaller. The lowest shrinkage was observed with the soil treated with 5,000 mg/L $CaCl_2$. A detailed discussion is included in the next Chapter, (Section 6.4.3.2). Time settlement curves and void ratio, effective stress results are included in Appendix E and F, respectively (Figures E8.1-E8.3 and Tables F8.1 and F8.3).

TABLE 5.13

SYNAERISIS SHRINKAGE RESULTS FOR MODIFIED CLAY SOILS

CELL #	SOIL	SETTLEMENT (mm)	PERCENT STRAIN
8.1	(25% bentonite + 0.009 M Ca)	0.960	3.32
8.2	(25% bentonite + 0.045 M Ca)	0.125	0.40
8.3	(25% bentonite + 0.005 M Ca)	1.518	5.47

In a MSW sanitary landfill, a gradual increase in the strength of leachate, rather than a sudden increase (see Chapter 2) is anticipated. It is extremely difficult to simulate the leachate generation and migration pattern in a synaerisis consolidation apparatus. However, the effect of variable leachate composition on clay-leachate interactions and the resulting internal stress changes, together with subsequent volume changes, could be easily demonstrated by changing the composition of the charging liquid in the leachate reservoir.

Cell C-8.4 was devoted to generate data to study the effect of leachate composition changes during the lifetime of a compacted clay. The 70% bentonite, 30% kaolinite sample was charged initially with the standard leachate (Ca^{+2} = 0.045M plus base). After 10 days of standard leachate permeation, the Ca^{+2} concentration was increased, first to 0.09M and then to 0.45M. A detailed discussion of the generated time settlement curve (Figure 5.9) is included in the next Chapter (Section 6.4.3).

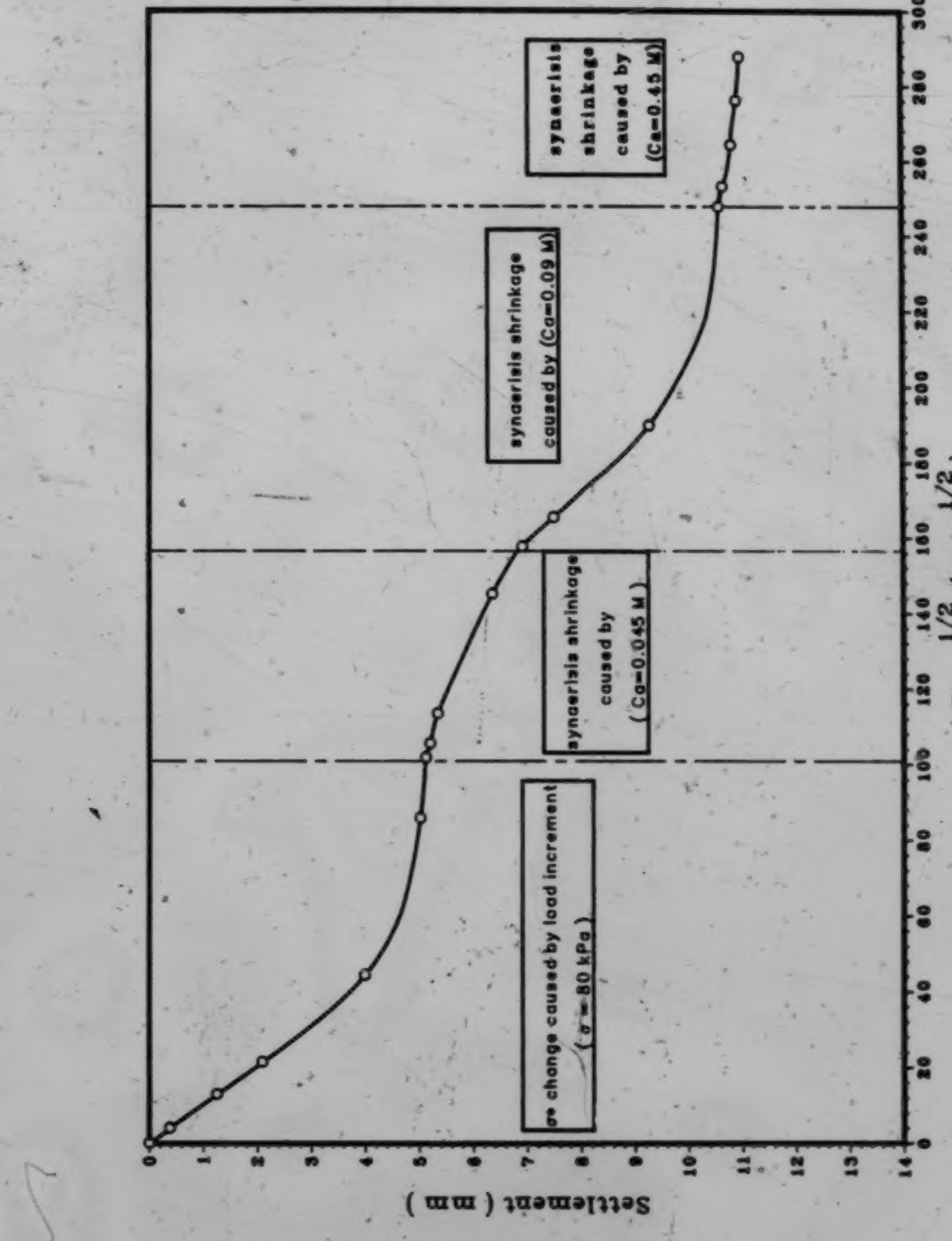

Figure 5.9 Time Settlement Curve for Cell C–8.4

5.3 SEDIMENTATION EXPERIMENTS

5.3.1 General Introduction

Throughout the experimental program, the sedimentation experiment was used as a tool for preliminary investigations. A large number of experiments were performed to select soils and chemicals before including them in the long term synaerisis consolidation program. The chemicals tested included, chlorides and sulphates of K^+, Mg^{+2}, Ca^{+2}, Na^+ Ba^+ and Fe^{+2}, organic solvents benzene and acetone, propionic, acetic and hydrochloric acids. Soils tested include pure clay minerals, native soils and a large number of mixtures of pure clay minerals (different combinations of bentonite/kaolinite and bentonite/Ca montmorillonite). Space does not permit presentation and discussion of all the results, therefore, some of the important results are discussed in this section and the rest are included in Appendix G.

As in the case of synaerisis consolidation, the process of sedimentation (floc formation and particle settling) is controlled by the interparticle physico-chemical forces. Therefore, the equilibrium sedimented volumes are indicative of the magnitude of the net interparticle repulsive force created between interacting clay particles. Higher swelling soils which are capable of developing high interparticle repulsive stresses or swelling pressures, sediment to a larger volume in very low (deionized water) electrolyte concentrations. Clay soils contain soluble salts. During soil sedimentation,these cations and anions are released to the sedimenting liquid. Therefore, it is not possible to sediment soils in a completely salt free environment.

The "free swell" volume (V_d), or the sediment volume with deionized water is useful in identifying potential swelling soils, or soils that develop high swelling pressures if compacted in a low electrolyte environment. Equilibrium sediment volumes, with deionized water, for several soils are presented in Table 5.14. These volumes are determined by the sedimentation procedure introduced in Chapter 4, using deionized water as the sedimenting liquid. As expected, larger sedimentation volumes were observed with bentonites. 10 mg of pure bentonite swell to a volume well above 100 mL and could not be determined in a sedimentation test in which a 100 mL graduated cylinder was used. Therefore sedimented volumes for bentonite/kaolinite and bentonite/Ca montmorillonites are reported.

A direct comparison of V_d values, obtained with bentonite/kaolinite and bentonite/Ca montmorillonite, indicates the negative effect shown by Ca montmorillonite on the swelling behavior of bentonite. In synaerisis consolidation testing, this phenomenon produced a smaller synaerisis strain. In sedimentation, Ca^{+2} injected to the sedimenting environment by Ca montmorillonites, encouraged rapid flocculation and formation of smaller sedimented volumes. A soil sample containing 40% bentonite and 60% Ca

TABLE 5.14

"FREE SWELL" (V_d) VALUES FOR SEVERAL SOILS

SOIL	EQUILIBRIUM SEDIMENT VOLUMES WITH DE-IONIZED WATER (V_d) (ml)
Kaolinite	18.0
Ca Montmorillonite	21.2
Bentonite - 70% Kaolinite - 30%	102.0
Bentonite - 70% Ca Montmorillonite - 30%	95.0
Bentonite - 40% Ca Montmorillonite - 60%	45.5
Bentonite - 40% Kaolinite - 60%	57.0
Bentonite - 33% Kaolinite - 67%	54.0
Bentonite - 33% Athabasca Clay - 67%	57.0
Athabasca Clay	24.0
Soil C	18.0
Soil C - 80% Bentonite - 20%	25.5

montmorillonite sedimented to a volume of 45.5 ml, whereas the sample with 40% bentonite and 60% kaolinite produced a sediment volume of 57.0 ml. Similarly, 70% bentonite produced sedimented volumes of 95.0 mL and 102.0 mL with Ca montmorillonite and kaolinite, respectively.

Results in Table 5.14 clearly indicate the sensitivity of the sedimented soil volumes to the type of clay mineral and the exchangeable cation type. In addition, the final sediment volume depends upon the type and amount of cations in the sedimenting environment. The sediment volume, being dependent upon the net physico-chemical interactive force between particles, is sensitive to any environmental change capable of changing these forces.

Different electrolytes or cation types affect the interparticle repulsive stress, R_1, to different degrees. If the soil, exhibiting a large free swell, V_d, or capable of maintaining clay particles far apart (large effective half particle distance, d), is sedimented in different liquids, the lowest volume is expected in the liquid capable of producing maximum reduction in the interparticle repulsive stress, R_1. Reduction in R_1, causes net interparticle stress to be more attractive, allowing particles to approach each other, effectively reducing the value of d. The final result is a smaller sediment volume.

Since a complete discussion of the sedimentation process is undertaken in the next chapter, this discussion will be limited to an analysis and discussion of the effect of type and concentration of cations on the sediment volume and an introduction of the proposed soil-leachate parameter, FSDP.

As noted earlier, both sedimentation and synaerisis consolidation processes (complete discussion is in the next chapter) are governed by the same phenomenon. Changes in interparticle forces, either as a result of changing the sedimenting environment or changing the characteristics of the pore water, causes the volume decrease in sedimentation and the synaerisis shrinkage. Preliminary results indicated a possible relationship between these two processes.

Therefore, to be amenable to direct comparison with synaerisis consolidation results, it was necessary to make use of a non-dimensional soil parameter, which could be easily derived from sedimentation test results. FSDP was used for this purpose.

FSDP was calculated using the following formula:

$$FSDP = \frac{V_d - V_l}{V_d} \times 100 \qquad 5.1$$

where,

V_d = Equilibrium sediment volume with deionized water,

V_l = Equilibrium sediment volume with test liquid.

The effect of increasing Na^+ and Ca^{+2} concentrations on the equilibrium sedimentation volume of a clay soil, containing 33% bentonite and 67% kaolinite, is shown in Figure 5.10. This figure, together with several other plots

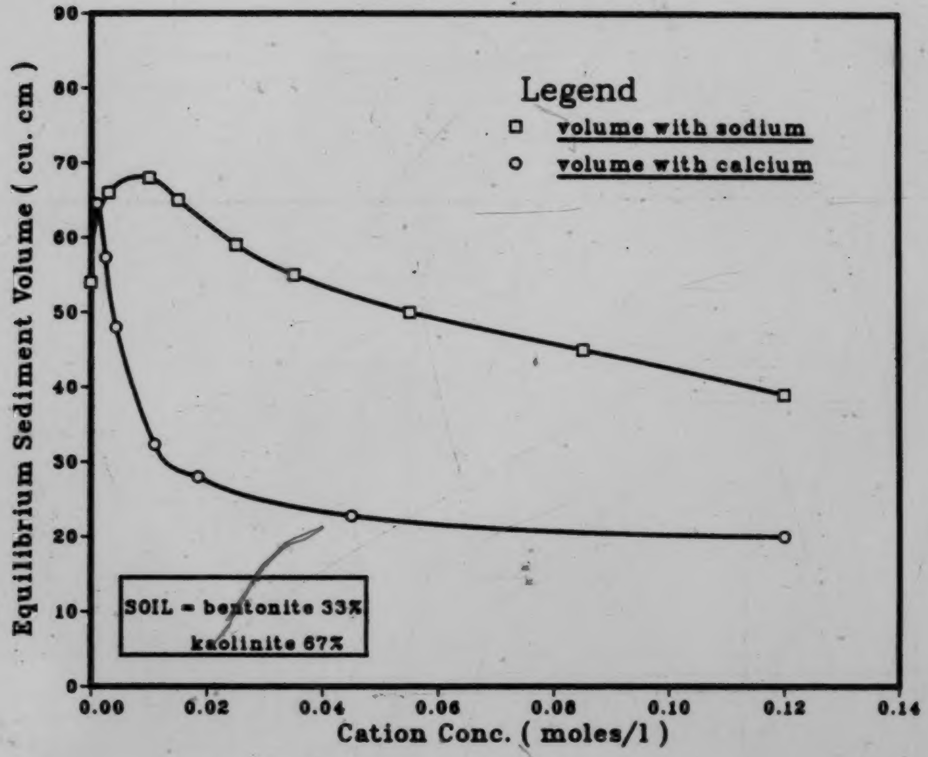

Figure 5.10. Equilibrium Sediment Volume vs Cation Conc.

developed for different cations, will be discussed in section 5.3.3. Figure 5.11 was developed from results obtained from a similar set of sedimentation experiments. In this case, the soil-leachate parameter FSDP was used as the dependent variable. A comparison of Figures 5.4 and 5.11 (independent variable in both cases being the Ca $^{+2}$ concentration) shows a close resemblance between the two plots. This provided evidence of a possible direct relationship between FSDP and percent shrinkage strain.

5.3.2 Correlation Between Percent Strain and FSDP

Although generation of percent strain values for a given soil and liquid combination takes several weeks (sometimes as long as 8 to 10 weeks), the corresponding FSDP value can be generated within 24 hours (although the cation exchange reactions, subsequent interparticle stress development and the resulting sediment formation were comparatively quick, at least 16 hours being necessary for the completion of the self weight consolidation phase of the sedimentation process). Therefore, it was decided to use the percent strain values already available (from series #1 to #5) and generate corresponding FSDP values for each soil and liquid combination. Figures 5.12, 5.13 and 5.14 show the correlation between percent strain and FSDP for the soils and liquid combinations used in series #1, #2 and #3. Figure 5.12 developed for a higher bentonite percentage, showed that a possible linear relationship exists between FSDP and

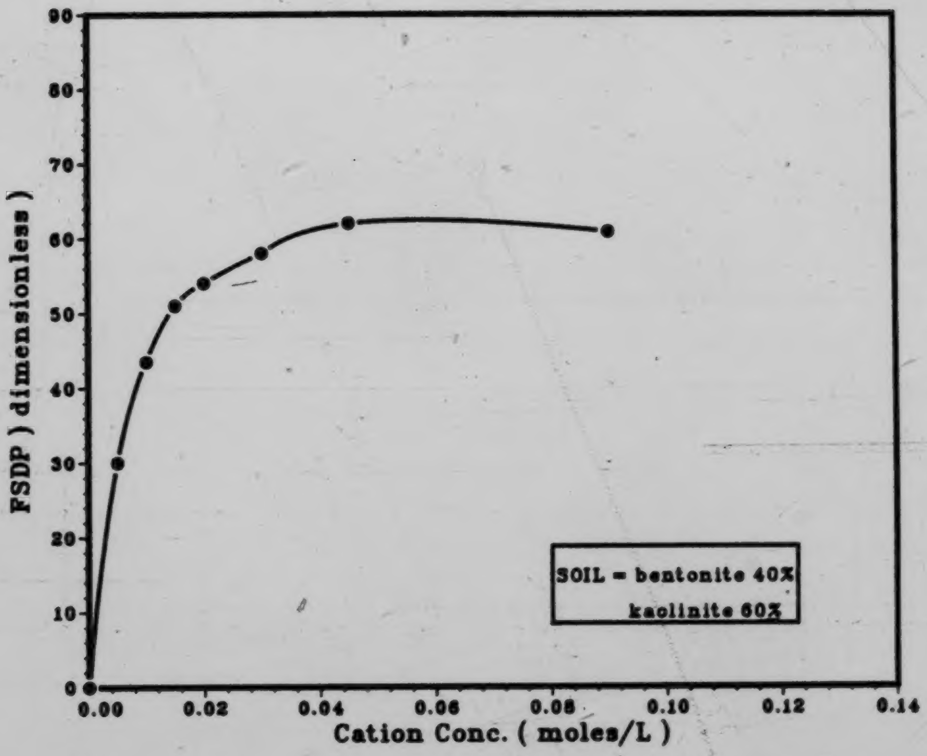

Figure 5.11. FSDP vs Calcium Concentration

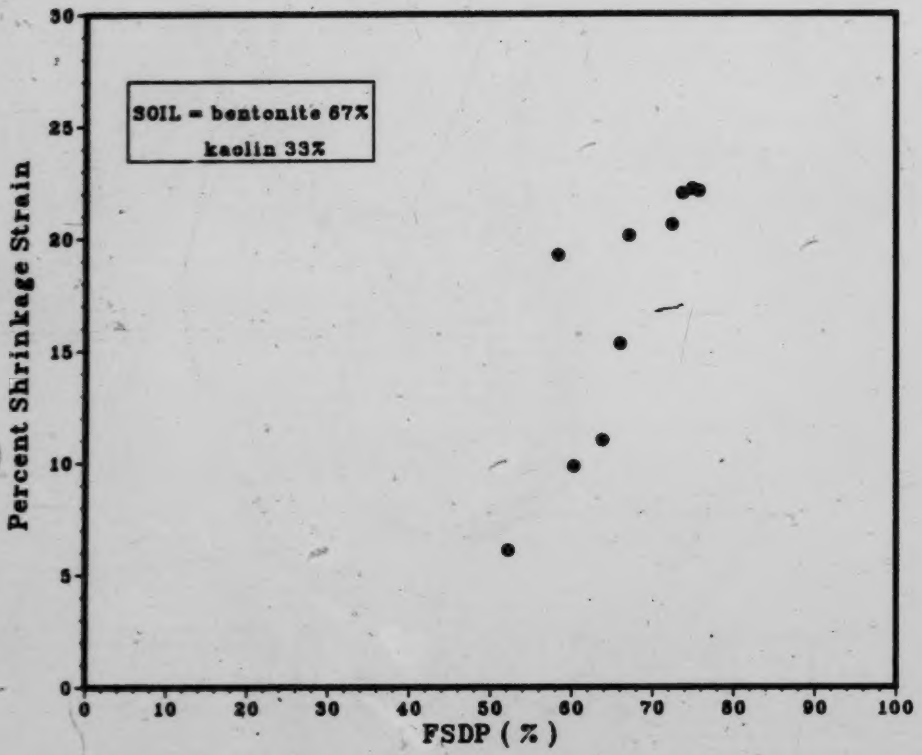

Figure 5.12 Percent Shrinkage Strain vs FSDP

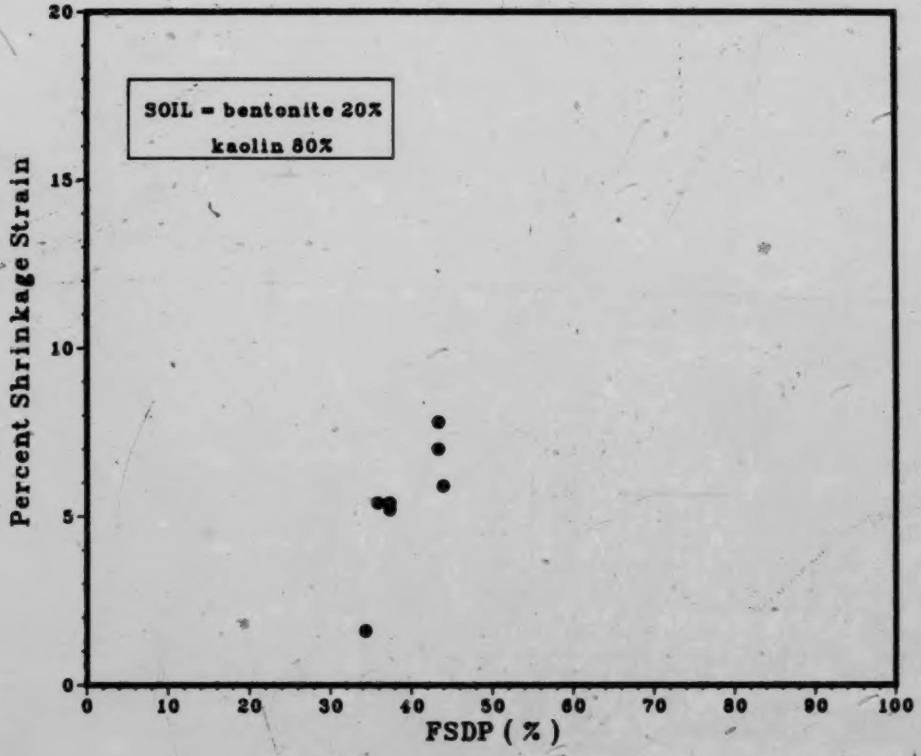

Figure 5.13 Percent Shrinkage Strain vs FSDP

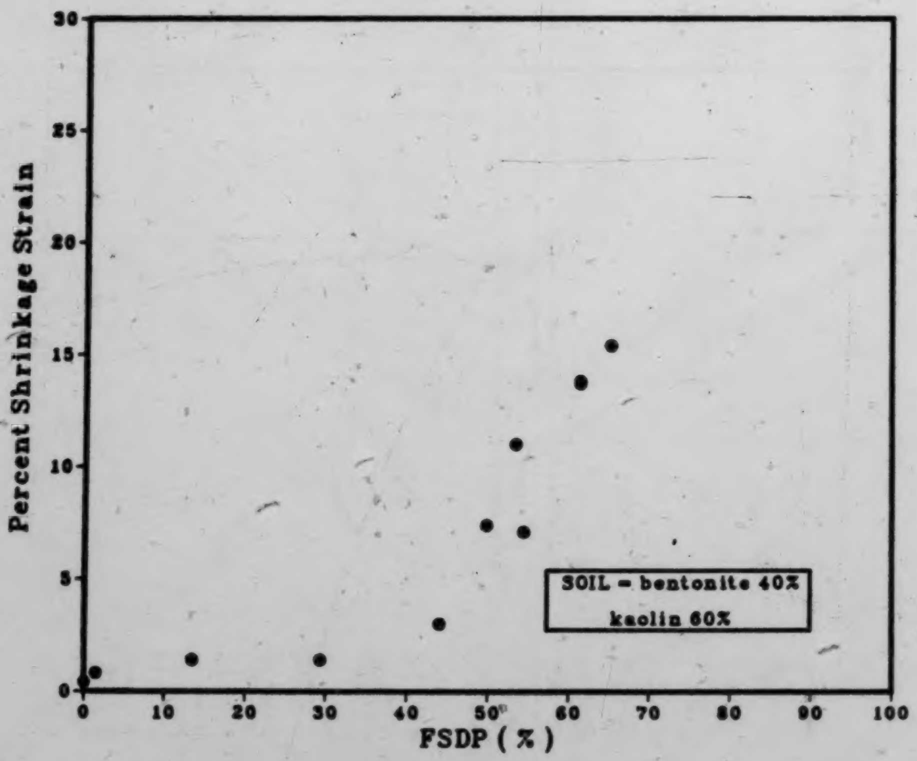

Figure 5.14 Percent Shrinkage Strain vs FSDP

percent strain. Point A, shown in Figure 5.12, would be clearly an outlier in such a relationship (NOTE: Figure 5.12 includes several percent strain values developed in later experiments).

It is worthwhile looking closely at the outlying point A in Figure 5.12. This point corresponds to cell C-1.3 (high acetic acid, low Na^+ and Ca^{+2}). As noted earlier, volume shrinkage in this cell may be caused by two mechanisms; physico-chemical stress changes and clay mineral dissolution. The magnitude of FSDP (or the sedimentation experiment) is unable to reflect the volume changes caused by clay mineral dissolution. This would be a significant limitation if the sedimentation test is adopted to investigate clay soils and low pH organic acids which are capable of dissolving the clay mineral. The functional relationship to be developed later will suffer in such a situation. However, this would not have a significant effect if the test liquid is MSW landfill leachate, with pH generally above 4.

Figure 5.14 (developed using series #3 results) clearly indicates the type of the relationship that exists between FSDP and strain. Figure 5.15 gives further evidence. In addition, Figure 5.15 shows that the functional relationship existing between FSDP and percent strain to be independent of the axial pressure applied during percent synaerisis shinkage determination. The three points, V1,V2 and V3, corresponding to the 85% bentonite/15% Ca montmorillonite

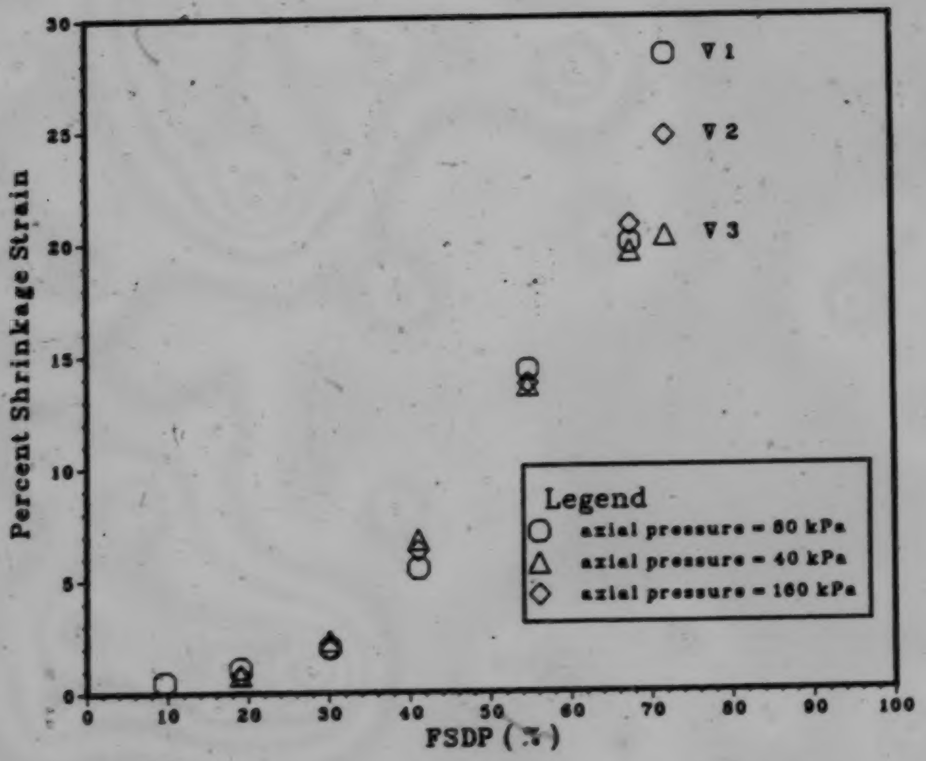

Figure 5.15 Percent Shrinkage Strain vs FSDP

cells are the exception. Higher unreliability of percent strain values at high bentonite content (as noted in Chapter 4) could contribute to the observed spread in percent strain values. Furthermore, very high percent bentonites are unrealistic in most field situations and such values were not given much consideration in developing the functional relationship.

Linear, least squares regression techniques were employed to elicit the correlation that may exist between percent strain and FSDP. All valid data points, excluding the three unreliable points discussed earlier (two points discussed here and the response observed with cell C-1.6, deleted because of corrosion problems) are included in the regression analysis. From the percent strains obtained for 3 different pressures (Figure 5.15), only one set of data (corresponding to 80 kPa axial stress) was included in the analysis. Initial regression and a residual analysis of percent strain versus FSDP demonstrated the need for a transformation. The scatter plot for all 38 points is shown in Figure 5.16.

Several transformations of the data points, including quadratic and logarithmic, were tried during the analysis. The percent strain needed to be subjected to a logarithmic transformation to account for the dependence of variance on this parameter.

There is a high degree of correlation (Multiple R = 0.964) between Log(percent strain) and FSDP. The regression

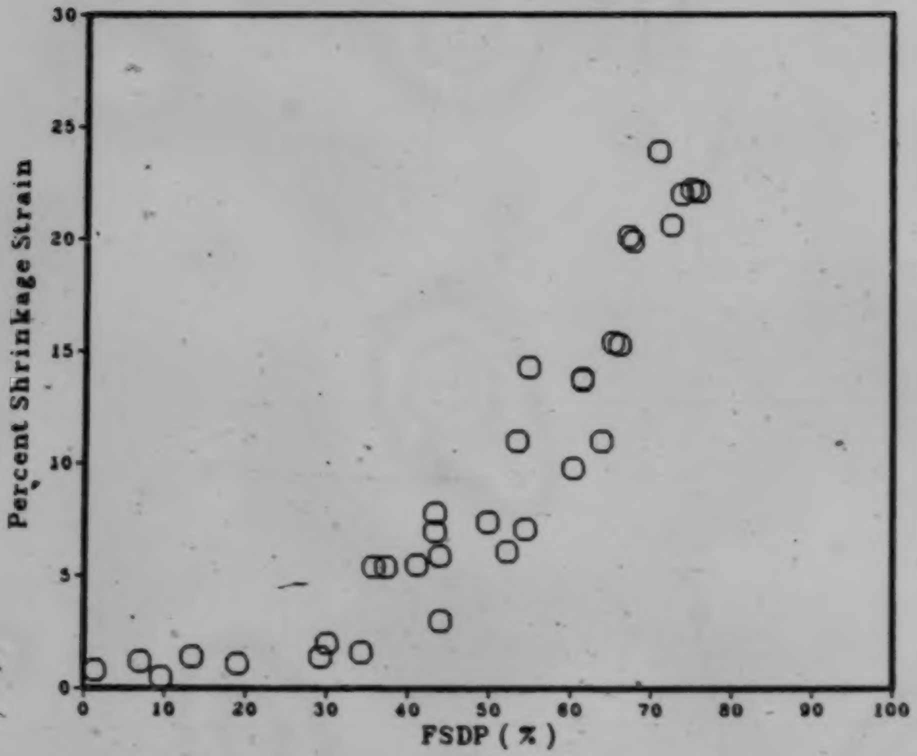

Figure 5.16 Percent Shrinkage Strain vs FSDP – Scatter Plot

analysis shows that 93.3% of the variation is explained by the regression. The resulting empirical relationship is:

$$\text{Log (percent strain)} = - 0.6582 + 0.052 \text{ (FSDP)} \qquad 5.3$$

The log transformation, managed to reduce the dependency of error variance on percent strain. However, the residual analysis noted higher variance at low predicted values. This may be due to the problem in reading small changes in volume during sedimentation. Therefore, it is recommended that a sufficient number of replicates be performed to minimize error at low FSDP values. The regression equation and the corresponding 95% confidence limits are shown in Figure 5.17

The empirical relationship developed here (Equation 5.3) is useful in predicting the expected synaerisis shrinkage from the FSDP values developed in a sedimentation test. The relationship holds good for a vast range of soil materials, exhibiting high or very low swelling potentials and correspondingly large or small FSDP values. However, it should be remembered that FSDP is only capable of predicting synaerisis shrinkages. Shrinkage caused by external forces or clay fabric disruptions, as a result of clay dissolution, is not reflected in FSDP values developed for the given soil and leachate. As will be discussed in the next section, it is possible to develop FSDP values for organic solvents. Because of the reasons cited earlier, high strength organics

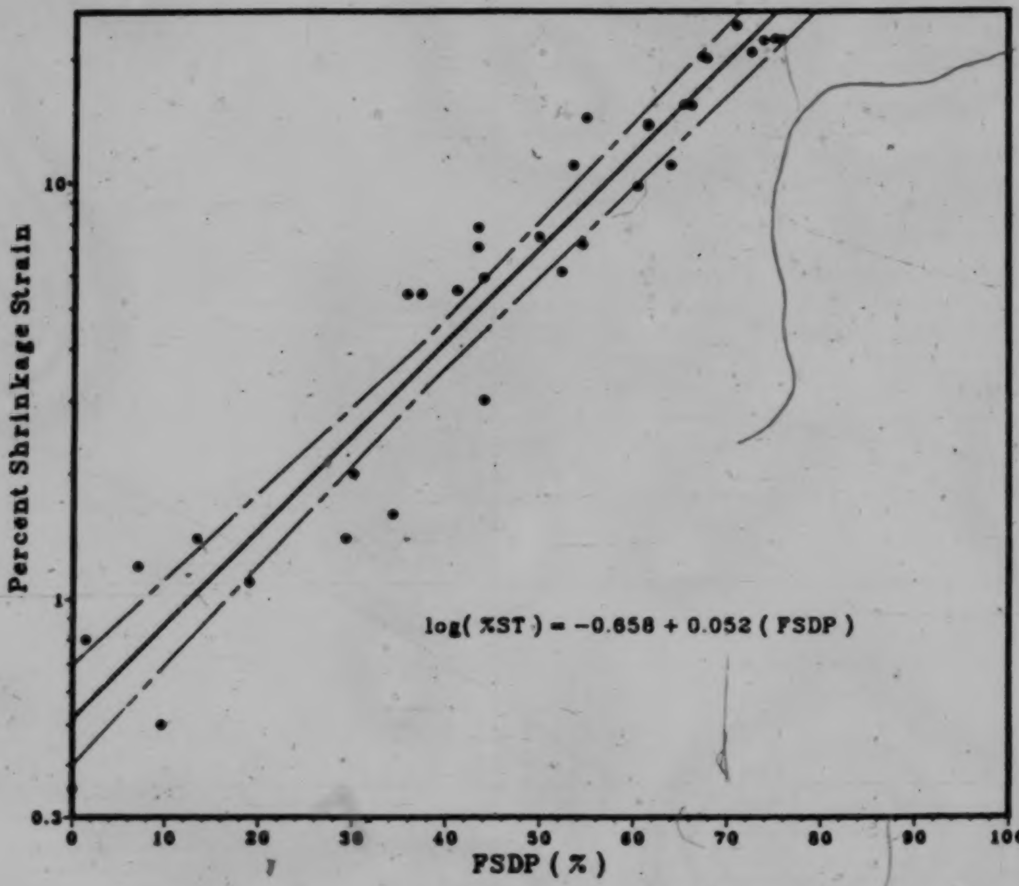

Figure 5.17 Percent Shrinkage Strain vs FSDP

were not tested by synaerisis consolidation procedures. Therefore, the validity of Equation 5.3, in predicting shrinkage caused by organic solvents, could not be verified.

5.3.3 Relationship Between FSDP and System Variables

Appreciating the possible use of equation 5.3 for predictive purposes, a large number of sedimentation experiments were performed on different soils with various electrolyte mixtures.

Figure 5.10, presented earlier, shows two of these plots. Na^+ and Ca^{+2} concentrations are plotted against the equilibrium sediment volumes read directly from a sedimentation test. Increasing Ca^{+2} and Na^+ concentrations reduced the equilibrium sediment volumes in both cases, but Ca^{+2} showed a significantly higher volume reduction. The corresponding FSDP values for increasing Ca^{+2} concentrations are plotted in Figure 5.11.

Figures 5.18 and 5.19 show sediment volume curves developed with 33% bentonite and 67% kaolinite, using Mg^{+2}, K^+ and acetic acid.

Behavior of Mg^{+2} was very much similar to Ca^{+2}. The slight difference (Mg^{+2} showed a larger sedimented value than Ca^{+2} for a given concentration) could be accounted for by the hydrated ion radius considerations. Hydrated ion radius of Mg^{+2} is higher than Ca^{+2} (10.8 and 9.6, respectively). Although the number of ions required to satisfy the negative charge in the clay particle is the same

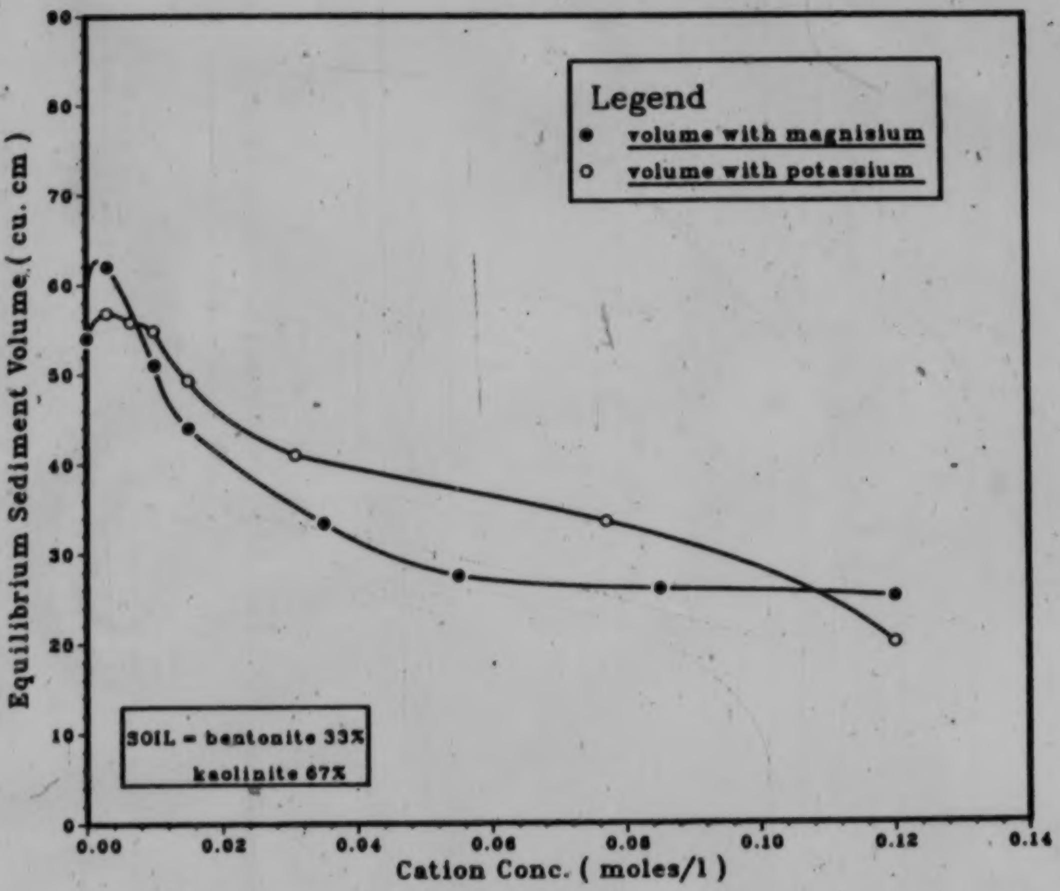

Figure 5.18. Equilibrium Sediment Volume vs Cation Conc.

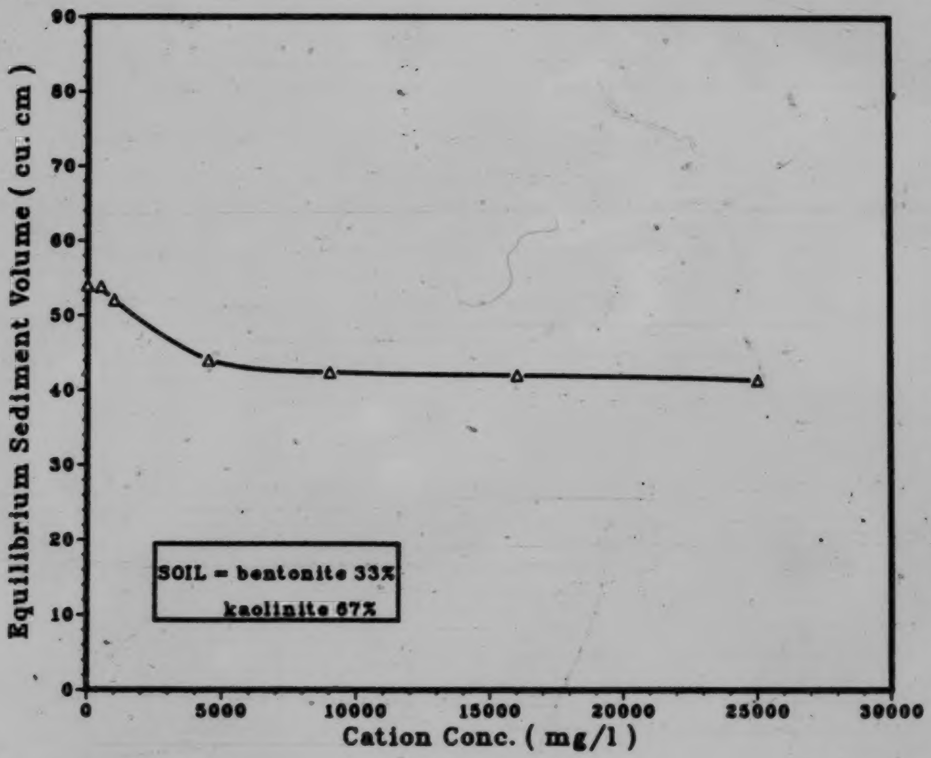

Figure 5.19. Equilibrium Sediment Volume vs Acetic Acid

for both Mg^{+2} and Ca^{+2} ions, the Mg-clay micelle will be larger than a Ca-clay micelle. Therefore, the resulting d values, and the corresponding void ratios for particles sedimented in Mg^{+2} solutions, are higher than that of Ca^{+2} solutions; this produces a sediment with a higher void ratio.

The monovalent cation, K^{+}, behaves more like Ca^{+2} or Mg^{+2} than Na^{+}. The peculiar behavior of K^{+} is explained by its capacity to reduce the interlayer swelling potential of montmorillonite, by locating itself between an alumina sheet and a silica sheet (see Appendix B-1 for details). Potassium ions, being of the right size to fit into the hexagonal holes of the silica sheet, would increase the strength of the bonds between two sheets and prevent further interlayer expansion. The non-exchangeable K^{+} ions subsequently create a basic unit, more representative of less swelling illite than montmorillonite. Therefore, the sediment equilibrated in a potassium environment behaves more like the clay mineral illite, and produce a smaller sedimented volume.

Acetic acid, with lesser amounts of monovalent H^{+} ions, need very high concentrations (or very low pH) to produce shrinkages similar to Na^{+}. As noted earlier, high acetic acid concentrations, with very low pH values, cause the aluminosilicate crystal structures to dissolve. The end products of such a reaction (gas bubbles formed within the clay mass) were noted during sedimentation experiments. As mentioned earlier, these reactions occur at very low pH values (around pH=2.5) which are not representative of a MSW landfill leachate. Although, not directly relevant to this research, some sedimentation experiments were performed with organic chemicals.

The organic fraction of landfill leachates are known to contain large amounts of organic acids. Sometimes, more than 75% of the organic acids is acetic. However, propionic acid is also present in large quantities. Therefore, before selecting acetic-acid as the representative organic chemical, it was necessary to compare the performance of acetic acid with other organic acids. Propionic acid was chosen for this comparison. Sedimentation experiments performed with replicate samples showed that there is no significant difference between the observed sediment volumes for acetic and propionic acids. Two concentrations (5,000 mg/L and 25,000 mg/L) of the acids gave similar results (38.5 ± 1.1 mL for acetic acid and 38.0 ± 1.2 mL for propionic) with a two to one mixture of kaolinite and bentonite.

Green et al., (1979) observed solvent breakthrough, while performing direct permeability tests with pure organic solvents, acetone, CCl_4, benzene, etc. Soil mixtures containing two to one bentonite/kaolin were sedimented in pure acetone and pure benzene. Both soils produced very low sediment volumes, 18.2 mL and 16.0 mL for acetone and benzene, respectively. This demonstrates the very high capability of these two organic solvents in shrinking a soil mass. So, it is not surprising to observe breakthrough and bulk flow during permeation of clay soils with these solvents. However, when the same soil was sedimented in a 50:50 mixture of acetone and water, an entirely different phenomenon occurred. Soil dispersed as in the case of pore water and produced a much larger volume. In MSW leachates, if organic solvents are present, they are in very small quantities, water being the carrier of all the pollutants. In such a situation, these organics are incapable of causing any shrinkage.

A close look at the sedimentation experimental results presented so far, directly points to the following conclusions: FSDP is a function of system variables; percentage bentonite in the system (%B) and leachate characteristics. It was suspected that temperature may have an influence. But subsequent experiments performed at 25°C (temperature controlled room) and at 0°C (Table G 3-1), did not show a statistically significant difference between the means of 2 sets (with high and low concentrations of Ca) of 6 replicate samples (the theoretical interpretation presented in Section 6.2 further supports this fact). Further experiments were performed to see whether wall effects have any bearing on the observed sedimentation values. Experiments performed with 100 ml and 250 ml graduated cylinders (different x-sectional areas) produced identical results (Table G 3-2).

With sedimentation experiments, it was possible to perform a large number of experiments and obtain a large data base within a reasonable time to develop a functional relationship between FSDP and the system variables. Essentially, this could be a functional relationship between the net interparticle repulsive stress ((R-A) component of effective stress) and selected system variables.

As noted earlier, MSW landfill leachates can be represented by Na^+, Ca^{+2} and acetic acid. Rather than using a huge number of variables (eg: all possible cation types in leachates) only these three plus the percent bentonite were used in the developement of the empirical relationship.

Therefore, sedimentation experiments were performed to elucidate certain aspects of the relationship $y = f(x,\beta)$, where:

y = response = FSDP,
β = a group of empirical parameters, and
x = the set of experimental variables (%B, Na, Ca and HAc).

First, it was necessary to perform a 2 level screening factorial (2^4 = 16 trials) to discern the relative significance of each of these variables, and the interaction effect on the response, FSDP. Information pertaining to the design and the results are given in Table 5.16(a). Considering the limited objectives of this excercise a linear, least-squares regression model was fitted to these 16 points using MIDAS statistical package (Fox and Guire, 1976). The highly significant "swamping" effect of %B (the two levels of %B - percent bentonite used were 35 and 65), precluded the verification of the importance of the other three variables.

The analysis clearly indicated the non-linear effect on response of some of the variables, particularly the response of Ca^{+2}. To this point of the analysis, it had been assumed that the response surface (FSDP) was planar which may not necessarily be true. Therefore, it was decided to examine the true parabolic response surface by enlarging the number of levels of each factor. At this point, it was decided to drop HAc from the variable list, to obtain more information on the other three more important variables.

Examination of a non-planar, parabolic surface is not possible using two-level factorial designs. Therefore a higher level design was used. The possibility of performing a large number of experiments within a fairly short period of time, allowed for adopting a 4^3 (64 trials) design to study the response.

The main objectives of fitting a response surface to these data are, long term prediction for the general pattern of FSDP and an interest in the true pattern of the response FSDP to the leachate constituents Na^{+}, Ca^{+2} and percentage bentonite. Acetic acid concentration was maintained at a low value of 250 mg/L (pH around 4.0). The generated results for the 64 trials are included in Appendix G (some of them are summarized in Figures 5.20 and 5.21).

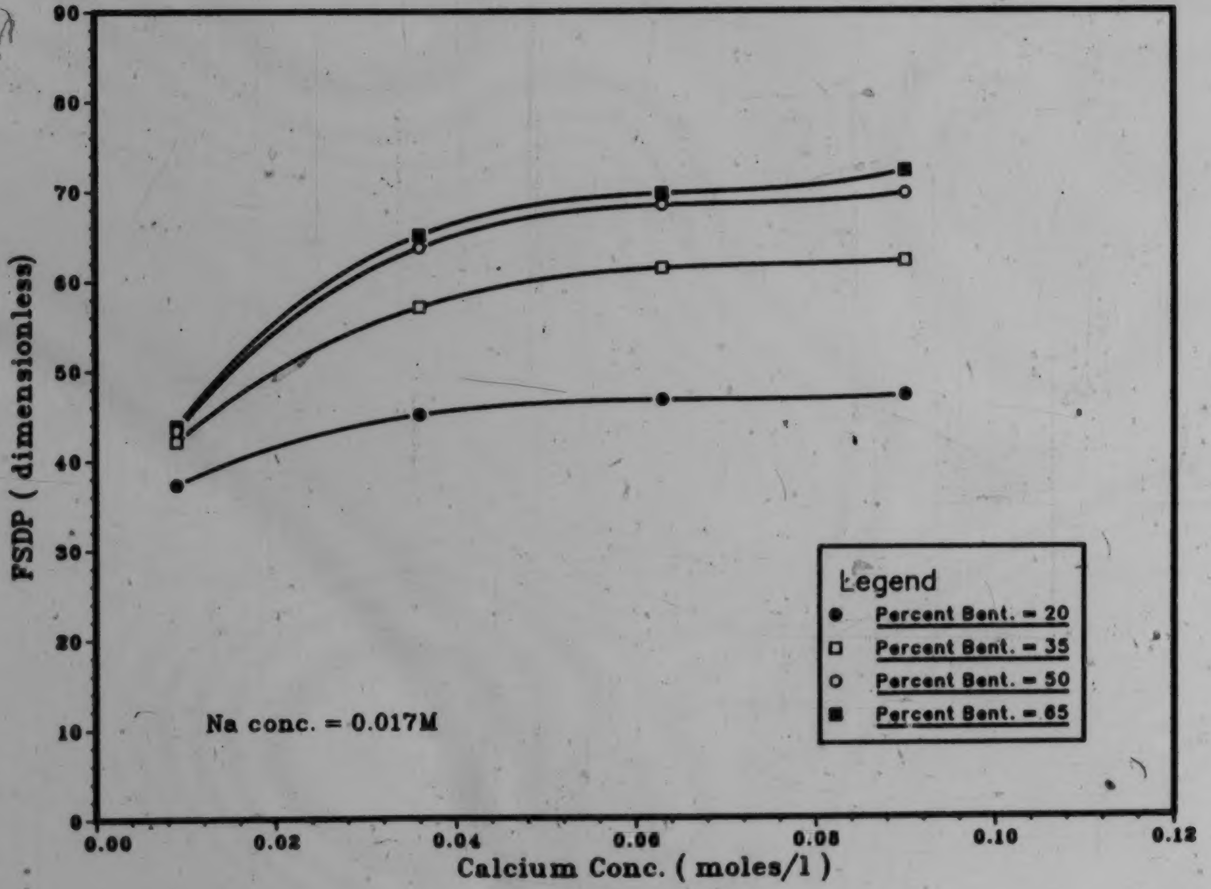

Figure 5.20. FSDP vs Calcium Conc.

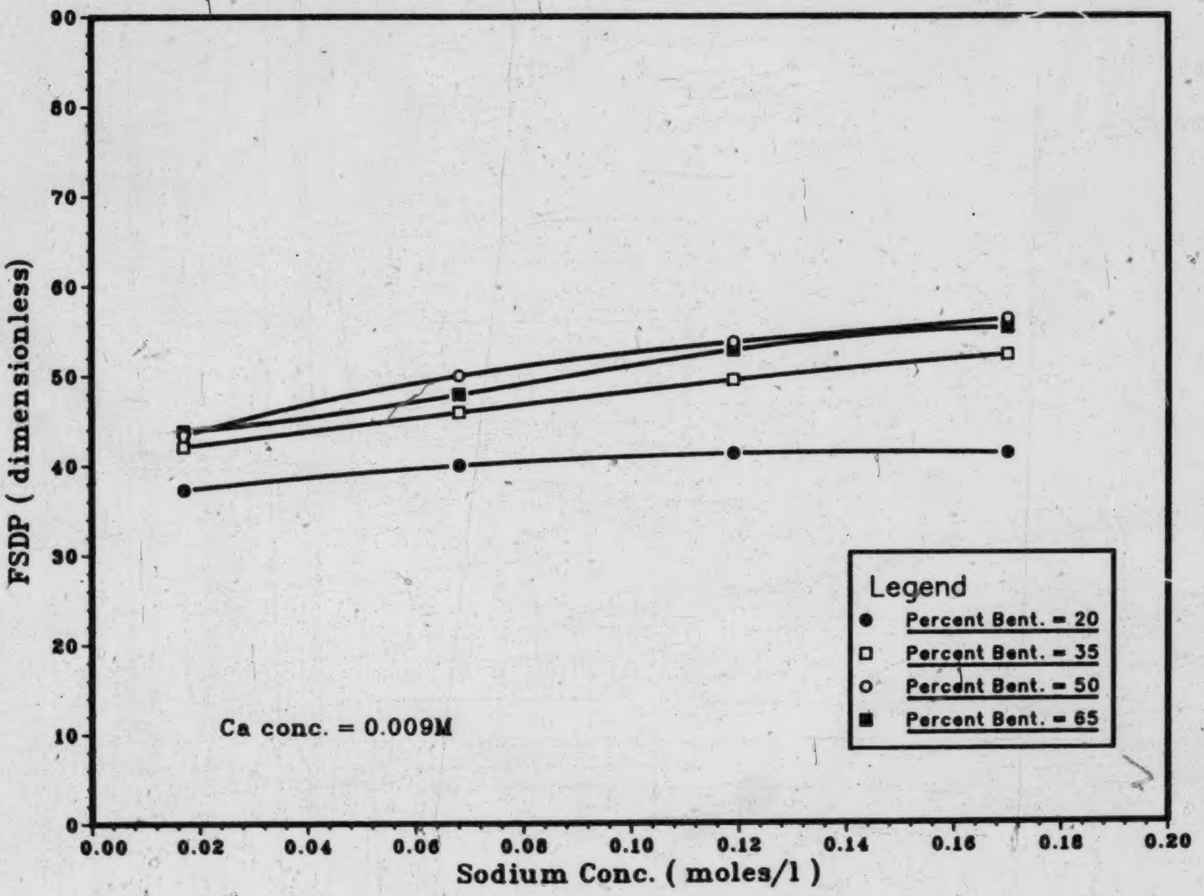

Figure 5.21. FSDP vs Sodium Conc.

Draper and Smith (1981) discussed the step-wise regression analysis as a method of selecting the most significant variables to describe a response surface. Using SPSSX statistical package and the forward regression technique, %B and Ca^{+2} ion concentration were found to be the most significant variables. This was consistent with the earlier findings reported, with both sedimentation and synaerisis consolidation experiments. Although less significant than the other two variables, Na^{+} was also included in a subsequently developed empirical relationship. Raw data needed to be transformed to fit the model. Although a high regression coefficient was achieved ($R^2 = 0.93$), it was necessary to perform diagnostic checking of the fitted model. Plotting and analysis of residuals were performed and outliers studied before accepting the following empirical model:

$$\log(FSDP) = 4.49 - \frac{10.18}{\%B} - \frac{322.9}{(Ca)} - \frac{71.66}{(Na)} \qquad 5.4$$

in which,

%B = percentage bentonite in soil,

Ca = Ca^{+2} concentration in mg/L,

Na = Na^{+} concentration in mg/L.

The estimated parameters were tested, by obtaining individual confidence intervals for each of them. They were found to be significantly different from zero at the 95% confidence level. During the regression analysis, the interaction effect $(Na^+) \times (Ca^{+2})$ showed some influence on the response. However, a subsequent check on the estimated parameter proved the effect on response to be minimal (β for $(Na^+) \times (Ca^{+2})$ interaction being equal to zero at 95% confidence).

As evident from equation 5.4, the effect of increasing %B, Ca^{+2} and Na^+ would be to increase the value of FSDP. The ANOVA table generated during the regression analysis showed that 68% of the log(FSDP) values predicted by this empirical equation will fall within the range Y' ± 0.0842 (Y' being the expected value).

5.4 HYDRAULIC CONDUCTIVITY AND COMPRESSIBILITY CHARACTERISTICS

Leachates with high cation concentrations are capable of changing the basic engineering properties of the soil materials (Chapter 3). These include hydraulic conductivity (k) and compressibility characteristics (as governed by consolidation coefficient C_v and compressibility coefficient, C_c).

5.4.1 Calculated Hydraulic Conductivity Results

C_c and C_v values, for a given soil, can be directly determined from the experimental consolidation curves and void ratio-log pressure plots. The same curves are used to calculate the soil parameter hydraulic conductivity (k).

The effect of leachate on C_c can be directly seen from a void ratio vs log pressure curve. C_c is the slope of the void ratio-log p curve. Figures 6.1 to 6.4, indicate that, C_c tends to increase after leachate attack, making the soil more susceptible to subsequent volume decrease, because of an axial stress increase (this subject is dealt with, in detail, in Section 6.1).

The effect of leachate interaction on C_v cannot be so easily appreciated. Based on the Terzaghi's theory of consolidation, it is possible to calculate the value of C_v using the time settlement data and the following equation:

$$C_v = \frac{T_v\, d^2}{t} \qquad 5.5$$

where,

T_v = time factor,

d = drainage path,

= layer thickness (H), for single drainage, and

= H/2, for double drainage,

t = time during consolidation.

The degree of consolidation by a soil, at any given time, depends on its time factor (Singh, 1976). For example, for the boundary condition of double drainage and overall linear distribution of consolidating pressure (as in this case), at 90% consolidation, T_v is equal to 0.848. The corresponding t will be t_{90}, which can be derived directly from a settlement vs $\sqrt{}$time curve.

Therefore, Equation 5.5 can be rewritten as follows:

$$C_v = \frac{0.848\ H^2}{4\ t_{90}} \qquad 5.6$$

Once C_v for a sample is calculated, the following equation can be used to calculate the hydraulic conductivity (k). In the equation,

$$k = C_v\ \gamma_w\ m_v \qquad 5.7$$

$$m_v = \frac{\text{change in volume}}{\text{(original volume)/(pressure increase)}} \qquad 5.8$$

The calculated k and C_v values for series #1 experiments are included in Table 5.15. Values calculated before and after liquid addition allowed the determination

TABLE 5.15

THE CALCULATED k AND C_v VALUES FOR SERIES # 1

CELL #	$C_v \times 10^{-8}$ (m^2/s)		$k \times 10^{-10}$ (m/s)	
	BEFORE	AFTER	BEFORE	AFTER
C-1.2	3.77	4.21	2.55	1.63
C-1.3	4.06	4.32	1.77	1.53
C-1.5	5.15	3.20	2.93	0.97
C-1.6	7.01	3.33	4.61	1.09
C-1.7	4.44	3.65	2.53	1.38
C-1.8	3.58	3.78	2.18	1.40
C-1.9	3.39	3.57	2.09	0.92
C-1.10	3.86	3.32	2.02	1.00
C-1.12	5.14	4.01	3.99	1.12

of the net leachate addition effect; this includes both physico-chemical and mechanical effects. Note the higher initial (at the point of liquid addition) void ratios observed for these samples.

Hydraulic conductivity (k) is a function of void ratio, and depends entirely upon the pore size distribution within a clay water system. At very low load increments, the void ratios were higher, and gave higher values for the hydraulic conductivity. Therefore, these values are not taken into account in calculating the average hydraulic conductivity before the leachate addition (initial k). The calculated k values approached a constant value in the pressure range 40 kPa to 80 kPa. However, fully appreciating the dependency of k value on the void ratio, the corresponding void ratios are also reported alongside the k values.

Since the variables that influence the hydraulic conductivity are both mechanical and physico-chemical, it is important to isolate the physico-chemical effects in understanding the leachate interaction effects. This is not possible for the series #1 results (because of the failure of the "control" experiment), but the comparative effect of each liquid combination is evident.

A lengthy discussion of effect of leachate on calculated k values is undertaken under the interpretation of experimental results (Chapter 6), and the interested reader is directed to Section 6.4.1.

5.4.2 Direct Permeability Test Results

In general, the calculated k values showed the hydraulic conductivity to decrease (marginally) as a result of the clay-leachate interaction. It was considered necessary to verify these findings in a direct permeability test. Since the practical application of permeability results were minimal, less time and equipment were devoted for this purpose.

Two soil samples, comprised of 20% bentonite and 80% soil A (passing #40 sieve), were prepared with deionized water. Initial permeability results were obtained at 80 kPa axial stress. Thereafter, soils were charged with standard leachate. Permeability measurements were taken (both at low and high gradients) regularly and used to calculate the hydraulic conductivity, k. Since these tests were performed with the modified consolidation apparatus it is possible to monitor the volume shrinkage caused by leachate.

Two synaerisis consolidation cells, were also loaded with 1:4 bentonite/kaolinite and 2:3 bentonite/kaolinite soil samples. Low gradient (ranging from 6 to 9) permeability tests were performed using 30% NaCl brine as the permeant. These results, together with the results obtained with standard leachates are analyzed and fully discussed in section 6.5.1.

5.5 MICROSTRUCTURAL INVESTIGATIONS

Four samples from series #5 were selected for the scanning electron microscopic study. (NOTE : Since the samples were required for SEM studies, the pressure increment 160 kPa, was not applied. Therefore, k values could not be calculated for the leachate affected soil samples).

Microstructural observations were made in two directions; parallel and perpendicular to the direction of axial stress application, on samples obtained from the following consolidation trials:

a) Trial # C-5.1 : bentonite and kaolinite at 60% & 40%, respectively. - with deionized water : $\sigma^* = 80$ kPa.

b) Trial # C-5.2 : bentonite and kaolinite at 60% & 40%, respectively. - with $Ca^{+2} = 0.045M$: $\sigma^* = 80 \text{ kPa} + \Delta\,(R\text{-}A)_1$,

c) Trial # C-5.3 : bentonite and kaolinite at 60% & 40%, respectively. - with $Ca^{+2} = 0.091M$: $\sigma^* = 80 \text{ kPa} + \Delta\,(R\text{-}A)_2$,

d) Trial # C-5.7 : bentonite and Ca montmorillonite at 70% & 30%, respectively. - with standard leachate, $\sigma^* = 80 \text{ kPa} + \Delta\,(R\text{-}A)_3$.

Photomicrographs thus obtained are included in Appendix H. The magnification is recorded on each photomicrograph.

Useful fabric information can only be obtained from representative samples, which retain their original characteristics during sample preparation. All drying techniques, including freeze drying, cause dimensional changes which may lead to fabric damage to samples, particularly to those which are delicate, such as sensitive soils (Bohor and Hughes, 1971). Clay soil samples, with a high content of montmorillonite, fall into this category, too. Their fabrics are so fragile, damage from sample preparation is almost unavoidable. Therefore, the micrographs shown in Appendix H should be discussed bearing these limitations in mind.

When moisture is removed from the clay soil sample, the increase in surface tension causes modification of the arrangement of clay particles. As noted in Section 4.5, freeze-drying involves rapid freezing and sublimation of the ice under vacuum, so that shrinkage caused by surface tension forces could be minimized. However, conversion of water into ice was accompanied by a volume increase of about 9 percent, which may cause localized stress accumulations.

Differential changes in dimensions, caused by thermal gradients, may also contribute to crack formation. Moisture movements which accompany freezing are another probable cause of fabric alterations (Gillot, 1980).

All these causes contribute to the formation of cracks within the liquid-Nitrogen-cooled soil samples. Almost all of the specimens prepared by this technique exhibited some degree of microcrack formation. Adaptation of a smaller specimen size and use of Freon as the coolant reduced, but did not eliminate, the size and number of cracks within the soil specimen.

Although initially the observed cracks were thought to be the result of synaerisis shrinkage, supporting evidence could not be acquired by a spot x-ray element analysis.

If the cracks were formed during leachate permeation, Ca^{+2} concentration in the vicinity of the crack should be significantly higher than that of the intact soil mass. As seen from the x-ray element plots in Appendix H (H2.1 to H2.4), no significant change in Ca^{+2} was found. In general, the samples permeated with higher Ca^{+2} showed some increase in Ca^{+2} concentration in the intact soil mass. However, nothing exceptional was found during the analysis of elements in various selected points within the soil system.

Identifiable differences were not observed between the soil fabrics of leachate treated and control soil samples. The soil fabrics, in general, could be categorized as semi-oriented arrangements. Although the preparation techniques favored such a situation, a completely dispersed or a perfectly oriented fabric was not found for the control soil sample.

6. INTERPRETATION AND FIELD APPLICATION OF EXPERIMENTAL RESULTS

6.1 GENERAL INTRODUCTION

Two types of soils were used in this study; pure clay minerals and natural clay soils. Pure clay soils consisted of the minerals montmorillonite, kaolinite and lesser amounts of illite. Each of these pure clay soils exhibited a unique response to a leachate environment. Simulated leachates containing varying amounts of electrolytes, affected the observed response to a great extent. The expected response was governed by the type and percentage of each clay mineral present, the exchangeable cations in the clay mineral, concentration and the type of cations used in the simulated leachate.

Understanding the behavior of the soil selected for the liner in the environment in which it is expected to function is important. The natural soil is a heterogenous mixture of sand, silt and clay particles and a variety of clay and non-clay minerals. The landfill situation at which the compacted liner should operate is complex and highly variable. Therefore, experiments simulating the field operating conditions and using the representative natural soils and leachate would not necessarily produce the results required to understand behavior in the field.

A major contribution to the volume change behavior, as studied during this research program, was provided by the clay sized particles. Use of pure clay minerals enabled the investigation of the fundamental nature of volume change arising from leachate interaction (synaerisis shrinkage) and allowed the establishment of a physical basis for the observed synaerisis shrinkage phenomenon. Consequently, the majority of the experiments were performed with pure clay minerals.

6.2 SYNAERISIS SHRINKAGE AND THE EFFECTIVE STRESS CONCEPT

Based on the synaerisis consolidation test results obtained with pure clay minerals, a physical basis of the synaerisis shrinkage phenomenon in clay soils and the factors controlling this behavior can now be established.

The volume change characteristics of soils and their shear strength behavior depend not only on the total stress applied to the soil, but on the difference between the total stress and the pressure established in the pore space (Chapter 3). The concept of effective stress, so defined, resulted in the following equation, for a saturated soil,

$$\sigma' = \sigma - u_w \qquad 3.4$$

The effective stress σ' controls certain aspects of soil behavior, notably the compressibility and shear strength.

However, this approach has been found inadequate in explaining the behavior of certain clay soils containing higher percentages of clay minerals, mainly the expanding lattice type clay mineral, montmorillonite. In these clay soils, besides intergranular pressures and pore water pressures, physico-chemical forces, R and A, play a major role in governing the engineering behavior. The following equation takes into account the forces created by the chemical environment of the soil-water system:

$$\sigma^* = (\sigma - u_w) - (R - A) \qquad 6.1$$

Contribution from the net interparticle physico-chemical stress, (R-A), to the true effective stress, σ^*, depends primarily on the amount of clay sized particles present in the system. In sandy soils, with negligible clay sized particles, contribution from (R-A) stress would be zero. On the other hand, in clay soils rich in expanding clay mineral montmorillonite, the contribution from a change in (R-A) stress to σ^*, therefore to the volume change behavior, would be a maximum.

Other factors controlling the (R-A) stress of a soil-water system, at a given axial stress, include the type of cation dominating the exchange complex of the clay mineral, pore water cation type and concentration.

During a synaerisis consolidation test, the chemical composition of the pore water was altered during the process of leachate addition. The effect of leachate addition, and the other factors mentioned above, can be seen from the results obtained in the synaerisis consolidation test.

At the beginning of the experiment, the clay particles in the soil slurry were seperated to a very high distance. Therefore, the soil samples containing higher percentages of high swelling (directly related to the particle separation) montmorillonites C-4.9, C-4.12, C-5.8, showed larger void ratios at the beginning of the experiment. Compared to the 12.90, 13.69, 12.70 void ratio values showed by these three, the sample containing only 15% bentonite measured at a void ratio of only 3.95. At this stage, the (R-A) stress would be minimal in all four soil samples. The swelling pressures (or repulsive stresses) developed during the initial consolidation process amounted to 85.23 kPa, 75.49 kPa and 99.84 kPa in cells C-4.9, C4.12 and C-5.8, respectively. At point C2 in Figure 6.3, the swelling pressure in cell 4.2 amounted to about 32 kPa.

The swelling pressures were calculated using the equations proposed in Section 3.3.1.4. Repulsive stress at point 2, R_2, is found from Equation 3.11 :

$$R_2 = R\ T\ C_0 \{ \frac{C_d}{C_0} + \frac{C_0}{C_d} - 2 \} \qquad 6.2$$

During swell pressure testing, reservoir water concentration has often been considered as the pore water concentration (Yong and Warkentin, 1966). The assumption that the concentration of cell water (in our case deionized water) is equal to the concentration of free pore water does not hold because of the soluble salts present in the soil. During sample preparation, the soluble salts diffuse in to the surrounding environment and increase the electrolyte content of the free pore water.

In chapter three, it was argued that the repulsive stress was created as a result of the interrelation between the double-layer water (between particles) and the free pore water (outside particles). Therefore, the concentration of free pore water, C_0, needs to be determined accurately in order to calculate the repulsive stress. Bolt (1956), considered the concentration of the equilibrium solution which is pressed out upon loading of the soil specimen, to be C_0. As noted in chapter four, this concentration is approximately equal to the soluble salt concentration determined by saturation extract procedures. Therefore, soluble salt data were used in determining C_0 for each of the soil samples. For samples C-4.9, C-4.12 and C-5.8, C_0 was taken as 0.0109 meq/L and for cell 4.2, it was 0.005 meq/L.

C_d, the cation concentration in the midplane between two interacting clay particles, was determined by using equation D.25. As shown from soluble salt data, in all 4 soils, the pore water cations are dominated by the sodium ions. Soluble salt data for clay minerals (3 samples contained 70-75% bentonite) presented in Table 4.6, shows that the bentonite contributed 14.52 meq/L Na^+ ions to the pore water as opposed to only 0.31 meq/L Ca^{+2} ions. Subsequently, the physico-chemical forces within the clay water system are determined by the Na^+ ion concentration. Therefore, it is appropriate to use a value of 1.0 for Z (cation valence) in Equation D.25.

$$C_d = \frac{\pi^2}{\beta Z^2 (d + x_0)^2} \qquad 6.3$$

The estimation of the half distance between interacting plates, d, has been discussed in detail by Bailey (1965). The estimation of d, from a knowledge of the water content at point 2 and the specific surface of the soil, is possible employing Equation 3.13, if the assumptions that the particles are of infinite extent and are equidistant throughout the homogeneous soil sample are made. These assumptions effectively eliminate any possible end effects, the presence of a dead volume and the occurrence of tactoids. Therefore, these assumptions may not hold true in many natural soil water systems which are heterogeneous in nature and may contain large amounts of materials which do not contribute to the interparticle physico-chemical forces. Even in pure clay mineral - high electrolyte (divalent) systems, direct application of Equation 3.13 would not be possible. Clay particles in such systems would not occur as individual particles, but would agglomerate and form tactoids. In such situations, allowance should be made for the loss of interacting water and "dead spaces" during d calculations.

However, the method of sample preparation and the low electrolyte content of the pore water provided an almost "ideal" situation at point 2 for the application of Equation 3.13. As seen from electron micrographs (plate H-2 for sample prepared with deionized water at an axial pressure of 80 kPa), the sample at point 2 (just before leachate addition) did not contain a completely homogeneous, parallel oriented structure. Although, all the conditions for such a soil structure were fulfilled (low electrolyte environment, high montmorillonite content, artificial sedimentation technique of soil preparation), some E-F bonding has occurred between positively charged kaolinite edges and negatively charged monmorillonite faces. The presence of small amounts of illite (about 10 percent) and K^+ ions (contributed by illite) and Ca^{+2} in the pore water, as notable in soluble salt data, would also contribute to the non-homogeneous soil structure visible in plate H-2.

Neither the double layer repulsion (R) nor the van der Waals attraction (A) can be exactly calculated for non-parallel (EF arrangement) particles. However, an estimation can be made from the equations available for parallel particles. Non-parallelism tends to increase the osmotic pressures, therefore the repulsive stresses, between particles. On the other hand, non-parallelism could increase the van der Waals attractive force too, and may cancel out the large increases in repulsive stress (Bailey, 1965). However, non-parallel particles may contribute to the "dead volume" (the volume of pore space that does not contribute to the interparticle), a concept that will be fully analyzed in Section 6.3.

Since it was not possible to determine the sample water content just before leachate addition, instead of Equation 3.13, Equation 3.12 was used with the calculated void ratio, e, and total specific surface of clay minerals, S. Since the dominant cation was sodium at this point, it was not necessary to modify the S value. The equivalent d values thus calculated, with the corresponding x_0 (distance from particle surface) value was used in calculating the C_d for each soil sample.

The magnitude of x_0 varies with the type of clay mineral and the cation type in the interlayer water. The approximate x_0 values for kaolin and montmorillonite are 2/Z and 4/Z Angstroms, respectively (Mitchell, 1976). In calculating repulsive stresses at point 2 (R_2), these become 2 and 4 Angstroms (since Z=1 for Na dominated soils), respectively. The determination of the value of x_0 becomes more critical at lower void ratios when the half distance is smaller. As seen from Table 6.1 for the soils C-4.9, C-4.12, C-5.8 and C-4.2 with larger d values, the contribution of x_0, on the calculated R_2 value, is small and the approximate x_0 values can be adopted. But at very low d values (in the range of 10 Angstroms), x_0 should be calculated using more accurate methods (Sridharan and Jayadeva, 1982).

At very low C_0 values, Equation 3.11 can be rearranged as:

$$R_0 = R\,T\,\left(C_d + \frac{C_0^2}{C_d} - 2C_0 \right) \qquad 6.5$$

In this case, with C_0 values ranging from 0.011 to 0.005M for all 4 soils, C_0^2/C_d could be considered negligible, and is removed from the above equation. Therefore, for the situation at point 2 (before leachate addition), Equation 6.5 reduces to:

$$R_2 = R\,T\,(C_d - 2C_0) \qquad 6.6$$

TABLE 6.1

CALCULATED HALF SPACING (d) AND REPULSIVE STRESS AT POINT 2, FOR SOILS C-4.9, C-4.12, C-4.2 & C-5.8

SOIL	HALF - SPACING (d - in°A)	REPULSIVE STRESS (in kPa)
C-4.9	39.80	85.23
C-4.12	41.25	75.49
C-4.2	65.70	31.66
C-5.8	37.66	99.84

The soil samples with higher bentonite percentages showed larger swelling pressures as compared to soil C-4.2, which had only 15% Bentonite.

The contribution from leachate addition to the shrinkage-pressure behavior of soil C-4.9 is shown by the A2-A3 portion of the curve, in Figure 6.1. Soils used to develop Figures 6.1 and 6.2 are similar in soil composition; but soil C-4.9 (in Figure 6.1) has been charged with the high concentrated (Ca^{+2} = 0.045M) leachate, and soil C-4.12 with the low concentrated (Ca^{+2} = 0.018M) leachate. A 29.4% void ratio reduction was achieved with C-4.9, compared to 13.4% with C-4.12.

The reduction in the void ratio was caused by the loss of water from the soil-water system. The charging of the soil with leachate causes a reduction in the interparticle repulsive stress. In soil C-4.9, the repulsive stress diminished from 85.23 kPa at point A2 (Figure 6.1), to a low 6.2 kPa at point A3; in soil C-4.12 the corresponding reduction was from 75.49 to 17.11 kPa. Equation 6.1 for the initial condition (point 2) will be,

$$\sigma_2^* = \sigma' - (R-A)_2 \text{ and,}$$

after charging with leachate, at point 3,

$$\sigma_3^* = \sigma' - (R-A)_3$$

Therefore, the net change in physico-chemical stress will be

$$\sigma_3^* - \sigma_2^* = (R-A)_2 - (R-A)_3 \qquad 6.7$$

Application of Equation 6.7, to cell C-4.12 produced,

$$\sigma_3^* - \sigma_2^* = (85.23) - (6.2) = 79.03 \text{ kPa.}$$

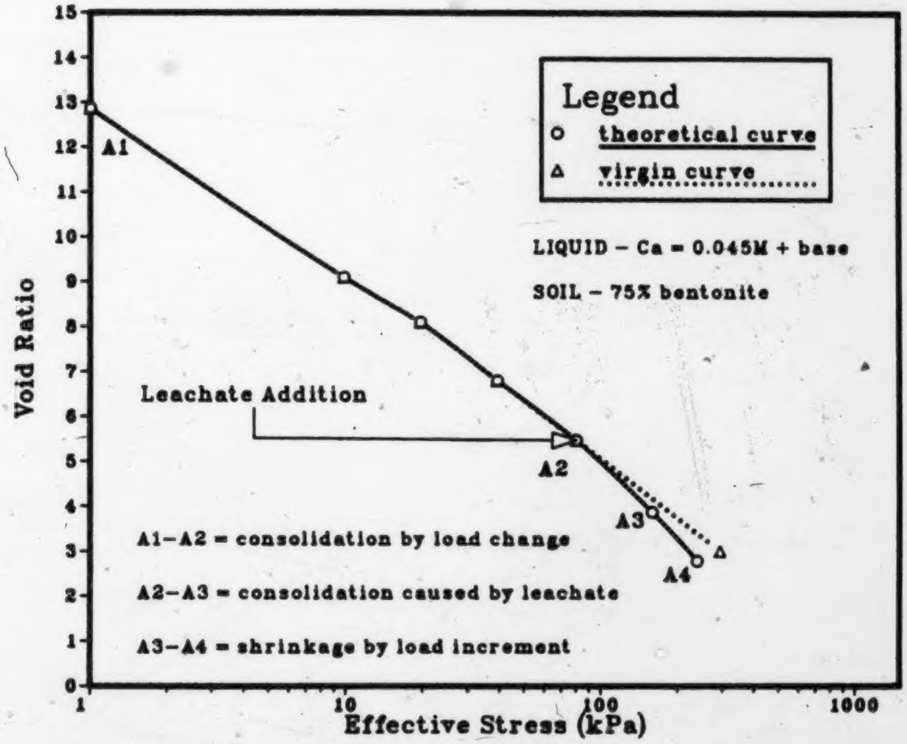

Figure 6.1 Void Ratio – Log Effective Stress, Cell C–4.9

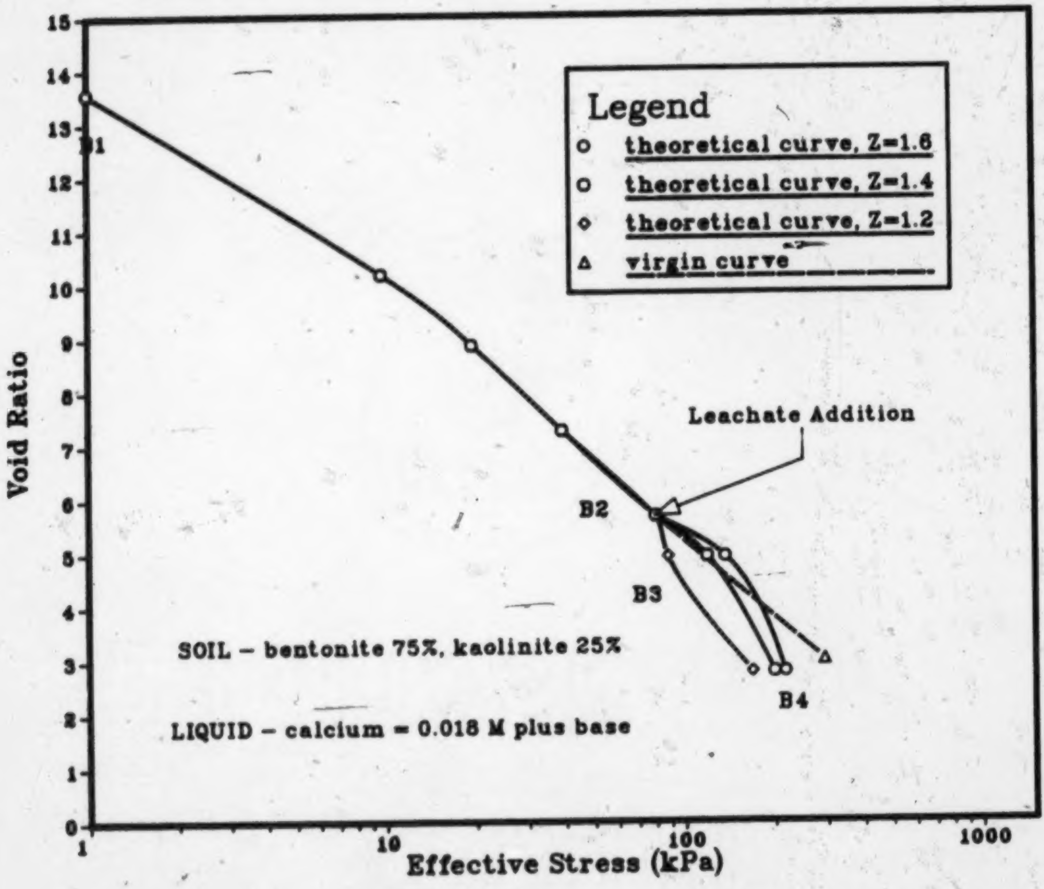

Figure 6.2 Void Ratio – Log Pressure for Cell C–4.12

Therefore, for C-4.9 and C-4.12, the reduction in net repulsive stress ((R-A) stress) is 79.03 and 58.38 kPa, respectively.

The increase in true effective stress, created by the leachate addition, causes the decrease in void ratio or "synaerisis shrinkage". The magnitude of volume decrease in shrinkage is directly proportional to the increase in true effective stress caused by the diminution of repulsive stress. The deviation from the straight line void ratio - log $\sigma*$ curve may be due to the inherent assumptions in the double layer theory (Figure 6.1). However, the calculated (based on experimental void ratio observations) curve seems to fit reasonably well with the expected linear virgin curve.

The double layer theory of particle interaction adequately predicts the measured swelling pressures for the high-swelling sodium montmorillonite at low salt concentrations (Yong and Warkentin, 1975). The validity of application of double layer theory, with regard to remoulded clays devoid of clusters or tactoids and cementation, was tested with reasonable success by Sridharan and Jayadeva (1982). However, as noted by Delage and Lefebvre (1983), the relation between the elementary clay particles and the macroscopic behavior during compression would be difficult to investigate, without consideration of intermediate structure levels, whenever tactoids and domains are encountered. If the clay structure is constituted by aggregation of particles, large discontinuities would occur which were not governed by the double layer phenomenon.

Once charged with leachates containing high Ca^{+2} concentrations, Na^{+} ions in the exchangeable sites would be replaced by Ca ions. The replacement takes place according to the Gapon equation (Eq. 3.2). From the works of Norrish (1954), Blackmore and Miller (1961) and others, it has been established that montmorillonite, with Ca^{+2} as the exchangeable cation, do not swell beyond approximately 19 Angstroms (with a d of about only 10 angstroms), even in distilled water. The cation Ca^{+2} is mainly present in the Stern layer, not in the diffuse ion layer and contribute to the stabilization of particles into tactoids or domains. The presence of Ca^{+2} in the exchangeable complex causes aggregation of particles and creation of tactoids. Therefore, there is an uncertainty as to the relevant surface area, that should be used in respulsive stress calculation. Since the particles are arranged in packets, repulsive forces could only act between the tactoids. An estimation of this external surface area could be made from the knowledge of the number of particles associated with each tactoid.

A corrected value for specific surface area, based on the assumption of 4 particles per tactoid, was used in calculating the repulsive stress at point 3 (R_3).

For cell C-4.9, the presence of particles as tactoids reduced the water content by 16.7%. On the other hand if 8 particles per packet were assumed, the water content reduced by 19% and decreases the R_3 by about 2 kPa (amounted to a $\sigma*$ increase of about 2.5%). Because of the higher swelling pressures (and high $\sigma*$ values) involved, the occurrence of tactoids did not affect the outcome of the e - log $\sigma*$ curve. This was also evident with the other 3 cells.

In Figure 6.1, the compression index (C_c) which is the slope of the linear portion of the e - log $\sigma*$ curve, seem to increase from 4.43×10^{-3} to 5.75×10^{-3}. The 30% increase in C_c is caused by the reduction in interparticle repulsion which generally tends to cushion the effect of an external total stress increase on the volume compressibility. In fact, the external stress is resisted and a part of it is carried by the interparticle physico-chemical forces as swelling pressure. This has strong implications in landfill situations, where a previously low or moderately compressible clay soil might become more compressible after leachate permeation. As noted in the unified soil classification table (Table 2.2) OH, CH or MH soils, with poor to very poor compaction characteristics, would consolidate fairly well after permeation with leachate. The improvements in compaction characteristics with leachate permeation would provide the basis for treatment of highly plastic clay soils (OH, CH or MH) during clay liner construction. This would be further investigated during the discussion of series 8 experimental results.

The difference in σ^* caused by the low Ca^{+2} leachate for cell C-4.12 was small. As expected from theory, the corresponding synaerisis shrinkage was smaller (as compared with cell C-4.9). The effect of the selected Z value on the calculation of R_3 is evident in Figure 6.2. From point B2 onwards the e - log σ^* curve took different directions depending upon the effective valency value. The liquid used in this case consisted of low concentrations of calcium (0.018M) and a base (consisting of 0.017M sodium and 250 mg/L acetic acid). This liquid mixture of equal strength di and monovalent cations needs to be considered as a mixed cation system, rather than a Ca dominated homionic system.

A review of the available literature revealed a large number of approaches to the application of double layer theory for mixed ion systems. However, the scope of this research does not allow an elaborate analysis of the system at hand, and therefore, an approximate value for Z_m was calculated using the following equation (Balasubramonion, 1972):

$$Z_m = \frac{\text{monovalent cation conc.(mg/L)} + 2\text{x}Ca^{+2}\text{ conc.(mg/L)}}{\text{Total cation concentration(mg/L)}} \qquad 6.8$$

In using the above expression, contribution from anions is neglected and it was assumed that the proportion of the monovalent to divalent cations in the midplane would be equal to that of the permeating liquid. Application of

Equation 6.5 to the situation in cell C-4.12 gave a Z_m value of 1.51. The validity of this simple calculation is seen in Figure 6.2, where the curve representing Z = 1.4 showed the best agreement with the virgin curve.

The effect of leachate on soil C-4.2, containing a lesser percentage of bentonite, is shown in Figure 6.3. Portion C2-C3 of the e - log $\sigma*$ curve, which followed the virgin curve, represents the 6% void ratio reduction as a result of the 25 kPa increase in true effective stress, $\sigma*$. Therefore, it was possible to consolidate the low bentonite, high kaolinite soil C-4.2, to a smaller initial void ratio (2.04). On the other hand, the corresponding initial void ratio for soil C-5.8, which contained 75% Bentonite, was 4.88. As in the case for soil C-4.9, soil C-5.8 showed a large volume reduction (a void ratio charge of 29.7%) during leachate permeation. Post leachate compression characteristics are not available (Figure 6.4) for this soil sample because the sample needed to be removed immediately after leachate treatment, for microstructural observations.

Calculated R_2 and R_3 for cell C-4.2 were 31.66 kPa and 6.62 kPa respectively. Therefore, the maximum possible effective stress change for cell C-4.2, due to leachate permeation, would still be only 26% of the effective stress change observed with cell C-4.9, for which $\sigma*$ change was, 95.4 kPa. Therefore, a comparatively small synaerisis shrinkage for C-4.2 was expected (Figure 6.3).

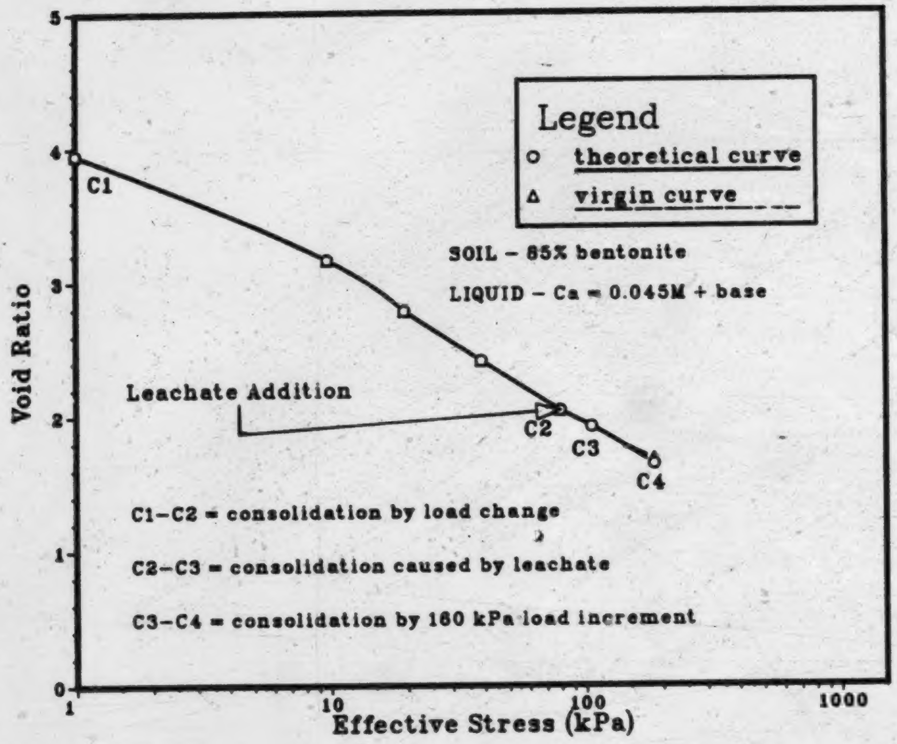

Figure 6.3 Void Ratio – Log Pressure for Cell C–4.2

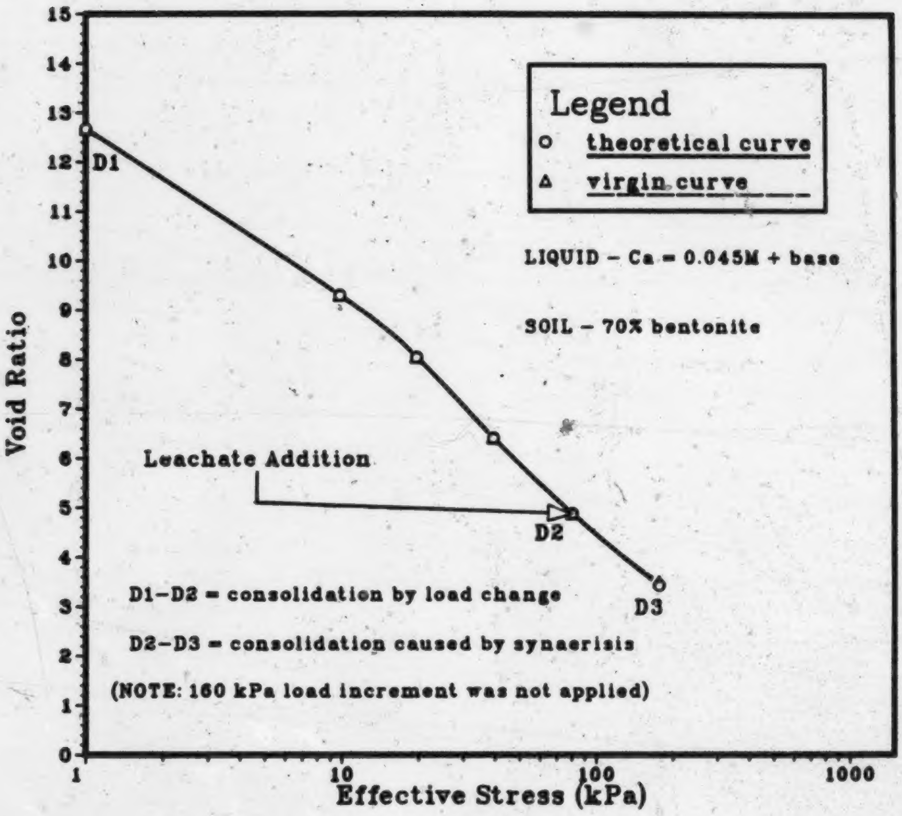

Figure 6.4 Void Ratio – Log Pressure for Cell C–5.8

Contribution from Interparticle Attractive Stresses

As noted in Section 3.3.2 and seen from Equations 3.13 and 3.14, the van der Waals attractive stress, which is the major contributor to the A stress, increases with decreasing interparticle half-distance d. As noted by Mitchell (1976), their contributions become significant only at very small half-distance values. Data reported by Sridharan and Rao (1979) indicated that the e - log p curves for pure clay minerals kaolinite and montmorillonite to be essentially the same, with or without the attractive pressure term. In this case, the pure clay mineral mixtures, behave in a similar manner. Furthermore, the interparticle half distances are very large, ranging from 37.66 to 65.70 Angstrom units. Therefore, the contribution from the van der Waals attractive stress should be fairly small. However, attractive stresses were calculated for all 4 soils selected for discussion herein and compared with repulsive stresses, R_2.

Equation 3.15 was used, with a value of 6.11×10^{-13} ergs, for the Hamaker constant, (used with success by Sridharan and Rao, 1979) and a montmorillonite particle thickness (δ) of 10 angstroms. The results are tabulated in Table 6.2 with the corresponding half spacing values.

Attractive stresses increased with decreasing particle distance, highest value being obtained at d = 22.7 for cell C-5.8. Largest contribution to the net physico-chemical stress (R-A) was observed in cell C-5.8. The magnitude of attractive stress change is 7.22 kPa, which is only 7.5% of the repulsive stress change caused by double layer diminution. The second largest attractive stress change occurred in cell C-4.9, which recorded an increment of 5.18 kPa (6.5% of the repulsive stress change).

By substituting calculated A values (Table 6.2), along with repulsive stresses for cell C-4.9, Equation 6.4 would produce the following results:

$$\sigma_3^* - \sigma_2^* = (85.23 - 0.71)\ \text{kPa} - (6.19 - 5.89)\ \text{kPa}$$
$$= 84.22\ \text{kPa}$$

This shows an increase of 5.18 over the previous value used in developing Figure 6.1. The 6.6% increase in true effective stress moves the theoretical curve marginally towards the virgin curve. Although the effect was marginal in this case, this clearly indicates the importance of incorporating an attractive stress term, particularly at smaller particle half distance (or smaller void ratio) situations. In Figure 6.5, the e - log σ^* curve for cell 4.9 is redrawn to include the attractive stress correction.

This situation will be further discussed during the analysis of synaerisis shrinkage studies at high surcharge pressure.

In summary, the results discussed here shows the general applicability of the double layer theory and the effective stress concept to explain the phenomenon of synaerisis shrinkage. The movement of leachate across a compacted clay soil liner has the ability to alter the

TABLE 6.2

CALCULATED HALF SPACING (d) AND CORRESPONDING ATTRACTIVE STRESS (A) AT POINT 2 & 3 FOR SOILS C-4.9, C-4.12, C-4.2, & C-5.8

SOIL #	HALF - SPACING (d in °A) at pt. 2	HALF - SPACING (d in °A) at pt. 3	ATTRACTIVE STRESS (kPa) at pt. 2	ATTRACTIVE STRESS (kPa) at pt. 3	CHANGE IN ATTRACTIVE STRESS (kPa)
C-4.9	39.80	24.40	0.71	5.89	5.18
C-4.12	41.25	32.00	0.58	1.61	1.03
C-4.2	65.70	58.40	0.68	1.61	0.48
C-5.8	37.66	22.70	0.85	8.07	7.22

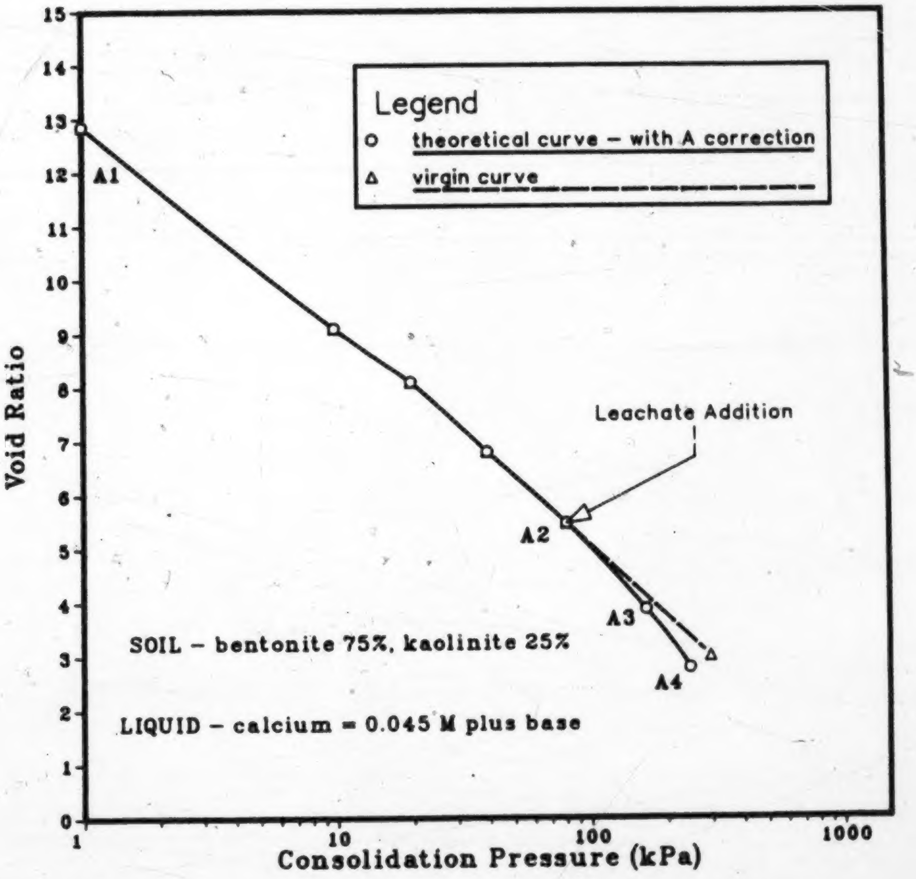

Figure 6.5 Void Ratio – Log Pressure for Cell C–4.9

interparticle forces which govern the post-compaction structure of the soil mass. The alteration in repulsive stress is much higher than the attractive stress, A. The change in net physico-chemical stress, (R-A) is controlled by such factors as the clay mineral type, type of exchangeable cations, concentration and the cation type available in the leachate. The importance of exchangeable cation type is evident in the results shown in Table 5.9, in which the effect of "standard leachate" on Ca montmorillonite/bentonite, kaolinite/bentonite mixtures of similar proportions were compared. In addition to being non-expanding, Ca montmorillonite tend to contribute Ca^{+2} ions (soluble salt data indicate 67% of soluble cations in this clay mineral to be calcium) to the pore water, thereby restricting the development of double layers within Na montmorillonite itself. As indicated by C-5.7, the addition of higher percentages of Ca montmorillonite to the bentonite tends to reduce the percent strain considerably (percent strain caused by standard leachate on the 95:5 Ca montmorillonite/bentonite mixture was less than 0.5%).

Osmotic Consolidation or Synaerisis Shrinkage

Turk (1974), studying leakage from solar evaporation ponds, noted that certain fine grained sediments originally placed in an environment less saline than the concentrated brine introduced into the evaporation ponds, are susceptible to subaqueous shrinkage, which forms vertical fissures in the fine grained sediment. The subaqueous shrinkage was thought to be the result of osmotic consolidation or the transfer of fresh water from the sediments to the more saline brines by osmotic action. He cited synaerisis shrinkage as an alternate mechanism of dewatering.

When two fluids of different ionic strength are separated by a semi-impermeable membrane, water molecules pass through the membrane but the movement of larger salt molecules are restricted. Interference from electrolyte molecules on the high concentration side diminshes the amount of water molecules allowed to migrate from the high electrolyte side, and hence, there is a net flux of water towards the high electrolyte concentration. A conceptual model of crack formation due to such action as proposed by Turk (1974) is presented in Figure 6.6.

Osmotic pressure differences caused by concentration gradients within a clay water electrolyte system could be as effective as hydraulic pressure gradients in causing water to move through a clay (Kemper and Evans, 1963). The flow resulting from osmotic pressure gradient depends upon the osmotic pressure gradient, the hydraulic conductivity of the

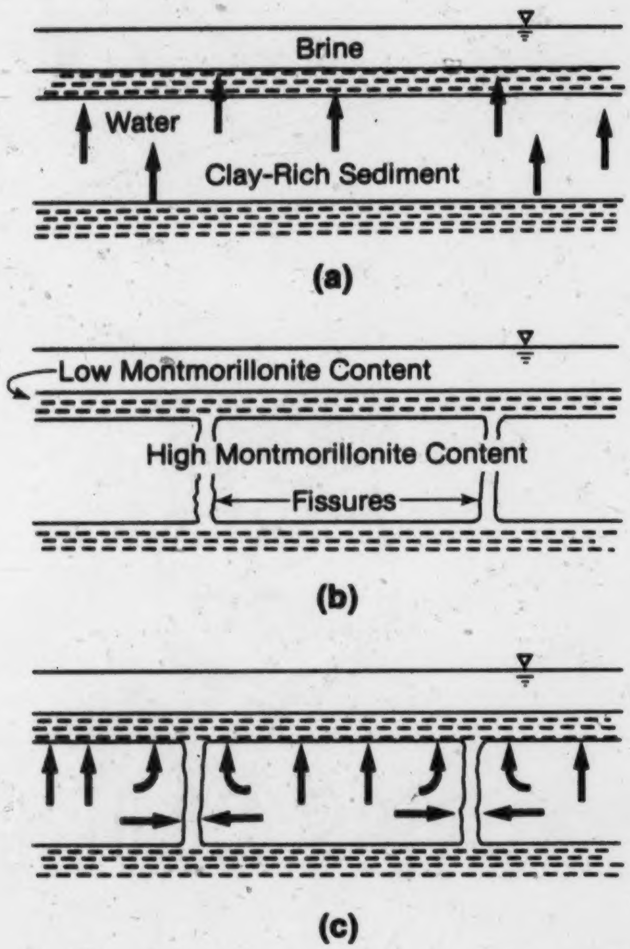
Brine
Water
Clay-Rich Sediment
(a)
Low Montmorillonite Content
High Montmorillonite Content
Fissures
(b)
(c)

Fig 6.6 PROPOSED MODEL FOR THE DEVELOPMENT OF FISSURES BY OSMOTIC CONSOLIDATION

a) Migration of Soil Water Towards Brine

b) Crack Formation Caused by Dehydration

c) Propagation of Cracks (modified from Turk, 1974) soil, and an osmotic efficiency coefficient (Kemper and Rollins, 1966). The coefficient relates to the degree to which the solute and solvent are differentially restricted by the porous medium. Once the surrounding liquid in the leachate reservoir is changed to the high concentration leachate, a situation is created for possible water loss from the soil sample to the surrounding liquid. It is interesting to see whether the observed shrinkage (caused by water loss) could also be explained by such a mechanism.

The osmotic flow and therefore the osmotic consolidation is independent of the hydraulic gradient. If osmotic consolidation is the major contributor to shrinkage caused by leachate, provision of a hydraulic gradient across the soil sample should not affect the magnitude of the response, ie. the two experiments performed with or without a hydraulic gradient should give the same shrinkage value. However, the two trials performed with identical soils and leachate (0.425M Na^+), with and without a hydraulic gradient, produced completely different results.

The bentonite/kaolinite 2:1 soil mixture, permeated with a solution containing 0.425M Na^+, at a hydraulic gradient of about 3, exhibited a total shrinkage of 4.6 mm in 3 weeks. On the other hand, the soil submerged in the same liquid, but without provisions for a hydraulic gradient, showed a total shrinkage of only 0.44 mm, in a similar time period. This shows the importance of diffusive flow and viscous flow of solute into the pore spaces, in response to hydraulic pressure gradients. If there is a major contribution from osmotic consolidation during shrinkage, it should be evident in the experiment performed without a hydraulic gradient.

An experiment performed with pure kaolinite and 0.09M Ca^{+2}, during the preliminary stages of the research program, gives further evidence to the claim that osmotic consolidation is not the major mechanisms causing shrinkage in test samples. The pure kaolinite samples prepared at 80 kPa axial pressure, showed only a very small shrinkage (0.06 mm) when tested with the leachate solution. If osmotic gradients created between the pore water and the leachate solution were important in these experiments, some amount of water should flow out from the soil sample. However, shrinkage results indicate that osmotic gradients have not caused substantial water flow out of the soil.

The results obtained during this research program provide clear evidence to suggest that the cracks observed by Turk (1974) were the result of synaerisis, rather than osmotic consolidation. The experimental evidence presented by Kemper and others (Kemper and Quirk, 1970), shows the presence of osmotic gradients within clay water systems; these contribute to movement of water to and from a clay soil mass. However, the contribution from such a mechanism, to the observed loss of water and shrinkage, is very much smaller than the contribution from effective stress changes caused by synaerisis.

6.3 SEDIMENTATION RESULTS

6.3.1 Sedimentation Behavior of Clay Soils

If left alone, the soil particles in a soil dispersion would "settle", forming a soil mass at the bottom of the container. The soil thus formed is then "consolidated" by its own weight. Imai (1980) identified three types of clay material settling. "Free settling" takes place in a low solid concentration soil-liquid mixture. The particle settling during this stage of settling is governed by Stokes' law. Behavior of thick dispersions of mineral grains and water, usually called "hindered settling" was described by Kynch (1952) and was studied in detail by McRoberts and Nixon (1976). During hindered settling, mutual interactions among the flocs restrict their free fall and the flocs settle in the aggregate with a unified rate; a sharp interface forms between the dispersion and clear water. The movement of the interface depends on the electrolyte content of the settling water. 10 g of soil mass (mixture of bentonite and kaolinite), shaken up with water containing 0.045M and 0.027M Ca^{+2} ions and allowed to settle, produced the plots shown in Figure 6.7.

In addition to the time settlement behavior (which appears to follow the pattern of consolidation time settlement curves), the nature and behavior of the soil sedimented at the base of the container seem to depend on the concentration of the Ca^{+2} ions in the sedimenting

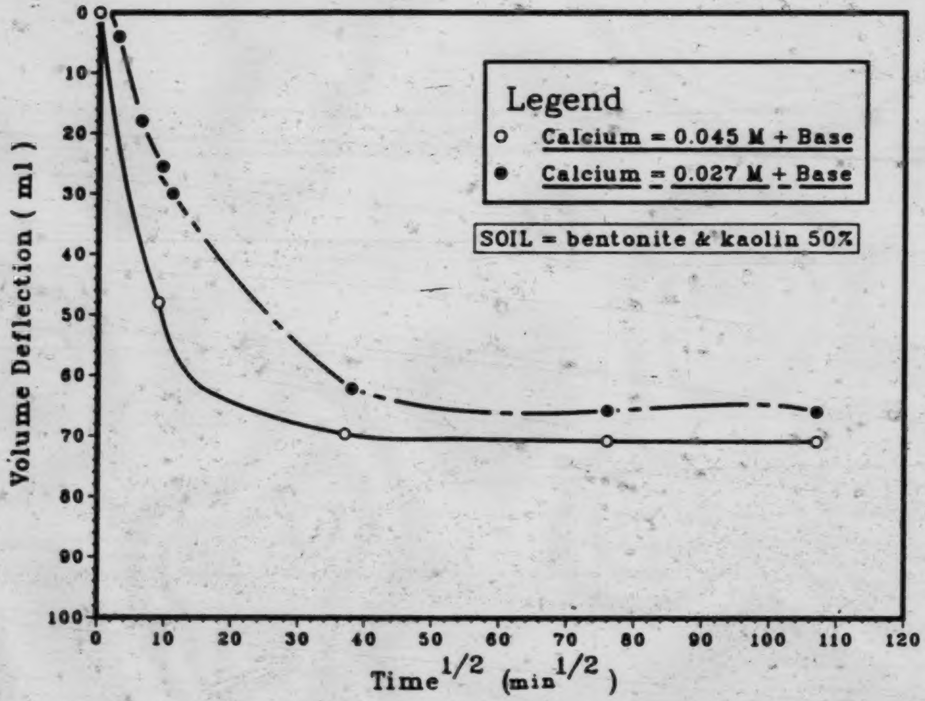

Figure 6.7 Sedimentation Curves

liquid. Higher Ca^{+2} concentrations produced smaller sedimented volumes.

The equilibrium sedimented volume in a sedimentation test was governed by the settling behavior and the subsequent "self weight" consolidation at the bottom of the container. When the soil is tipped on to the liquid surface, at the beginning of the sedimentation test procedure, the clay particles in the soil material will be subjected to physico-chemical forces. These interparticle interactions and subsequent floc formation and settling will be governed by the following three factors (Imai, 1980):

- the type of soil, especially the type of clay minerals present,
- concentration of solid materials,
- type and concentration of ions present in the sedimenting liquid.

In addition to these factors, the proportion and type of cations in the exchange complex, the soluble salts associated with the soil material would become important considerations.

The amount of solid material tipped in at one time was controlled during the experiment (see Section 4.5). Because of the low solid concentrations involved, the flocs formed at the beginning of settling are allowed to fall freely without hinderance from other flocs. The settling rates were governed only by the size of the flocs formed. Liquid viscosity became a factor only at very high salt concentrations. Since the final sedimented volume was indicative of the degree of flocculation and size of the flocs formed as a result of soil liquid interaction, it was considered unnecessary to study the hindered settling behavior of the soil. The study of hindered settling of aggregates also yields information on flocculation behavior of clay soils, but tight control of the experimental procedure would be essential to obtain reproducible results.

Despite being simple, sedimentation experiments yielded highly reproducible results (chapter 5). It should be remembered that this procedure was not developed to duplicate a field situation but was developed as an "index test", which gives results that are compatible with the long term synaerisis consolidation results.

The final sediment volume, obtained from a sedimentation test, was dependent upon the floc size and the degree of "self weight" consolidation and reduction of water content. The net interparticle repulsive stresses within the clay-water-electrolyte system affect both processes. As shown from results presented in section 5.3.2 the equilibrium sediment volume was a direct function of type and concentration of cations in the sedimenting liquid. Among the various cations found in large quantities in MSW leachates, divalent cations show the largest effect on the equilibrium sediment volume.

6.3.2 Mechanical and Physico-Chemical Force Systems Within a Sedimented Soil Mass

In order to develop a theory to account for the nature of the sedimented soil masses observed during sedimentation of clay soils in different electrolyte solutions, it is essential to understand the "self weight" consolidation behavior of soil solids, and the nature of the physico-chemical forces which govern the magnitude of equilibrium soil volume.

The soil particles or flocs, once settled at the bottom of the container, are brought closer to each other by the self weight of the soil. As additional material settles on top, the spacing between flocs or particles decreases still further, with the expulsion of pore water from between and inside the flocs (Been and Sills, 1984). Drainage occurs from the top of the soil mass, and consolidation, as a result of the self-weight of the soil particles, proceeds. At some stage, the sediment will have ceased to behave as isolated particles in a suspension, and will eventually become a soil with properties described by traditional soil parameters. This is the basis for the self weight consolidation theory as proposed by Imai (1981) and Been and Sills (1981).

6.3.2.1 Effective Stress Concept and Sedimentation

The settling column studies performed by Been and Sills (1981), on an estuarine mud containing 30% clay, showed that no measurable effective stresses are discernible in dilute suspensions. However, when the flocs in suspension come into contact during settling at the bottom, effective stresses begin to develop. Been and Sills (1981) identified a certain density at which a slurry can no longer exist in an effective stress free environment. The value of this density varies from soil to soil, sedimenting liquid characteristics and other factors.

Imai (1981) studying the consolidation stage of sedimentation of clayey soils, noted that, early in the consolidation stage, the self weight of the soil, which aggregate near the top surface, is compensated by the upward seepage forces induced by the flow of water squeezed out from the underlying sediment. In this stage, the effective stresses acting near the surface is almost nil, and the loose fabric of the soil aggregate remains near the surface. As consolidation proceeds and the seepage forces become weak, the self weight gradually turns into an effective stress. This increasing stress may first destroy the loose aggregate of flocs and then compress the flocs themselves. The sediment volume reduction proceeds until all the pore pressure developed within the soil has dissipated (Umehara and Zen, 1980).

As noted by Imai (1980), a discussion of "self weight" consolidation theory does not account for the physico-chemical aspects of sedimentation behavior. The dependency of the equilibrium soil volume on chemical characteristics of the liquid was shown clearly in Figures

5.9, 5.17 and 5.18. The forces acting within the sedimented volume would include its self weight (as noted above), surcharge pressure caused by the liquid column (negligible compared to other forces), and the net physico-chemical stress (R-A). The true effective stress (σ^*), for this situation would be,

$$\sigma^* = \sigma_s - (R-A) \qquad 6.9$$

The van der Waals attractive and long range repulsive forces between particles and self weight are only a few components of a more complex force system acting between particles (Scott, 1965). The results presented in Section 5.3 indicate the domination of the net interactive forces by the (R-A) component in clay materials. The situation which tends to reduce the net (R-A) stress (high electrolyte environment and low bentonite content) resulted in the smaller, sedimented soil volume.

When interparticle repulsion is high, R easily overcomes the (σ_s + A) stress and prevents the deposition of the material and creation of an effective stress. This is evident from the behavior of bentonite clay in distilled water, which behaves as a soil gel. Once the electrolyte content is increased, (R-A) stress is reduced, which invariably allows some degree of floc formation. The suspension starts settling and approachs a pseudo-soil condition, with a very large void ratio.

Repulsive stress is greater in a sodium montmorillonite (majority of clay mineral present in bentonite) system because osmosis exhibits a first-order relation to the number of ions present. An ion swarm of univalent sodium contains twice as many ions as does a corresponding swarm of divalent ions. Furthermore, because of its single valence, the Coulombic attraction of a sodium ion to the charged particle is less than that for di- or trivalent ions. Sodium ion swarms, therefore, range further from the particle surface and interact repulsively at greater interparticle distances. Sodium ions are capable of creating large repulsive stresses, which tend to push the particles apart and promote particle dispersion. In the sedimentation tube, these repulsive stresses easily overcome the opposing forces, self weight component (σ_s), overburden pressure from the liquid column and the van der Waals attractive force (A). The particles are pushed apart until a balance between all the forces within the system is achieved. The resulting system is closer to a clay suspension than a clay soil. For the clay suspension, with zero effective stress (Sills and Been, 1981), Equation 6.9 becomes:

$$\sigma_s + A = R \qquad 6.10$$

6.3.3 Sedimentation and Synaerisis Shrinkage

At higher particle separations (or at very high void ratios), clay minerals exhibit extremely low shear strengths and effective stresses (Been and Sills, 1981). The magnitude of the London-van der Waals attractive forces between particles would also be low. In Equation 3.15, at very high interparticle d spacings, the value of A becomes negligible. As an example, for a d value of 100 Angstroms and for the montmorillonite clay mineral with a Hamaker A value of 6.11 x 10^{-13} ergs, the attractive stress would be only 0.066 kPa. The pressure caused by the self weight of 10 g of clay material and the overlying liquid (less than 200 mm) will also be extremely low. Therefore, at larger void ratios obtained during sedimentation experiments, the left hand side of Equation 6.9 would essentially be equal to zero. Since,

$$(\sigma_s + A) = 0$$

it would follow that, for the sedimentation situation,

interparticle repulsive stress = R = 0

Since, R, T and C_o are not equal to zero, equation 3.11 would produce,

$$\{\frac{C_d}{C_o} + \frac{C_o}{C_d} - 2\} = 0 \qquad 6.11$$

Simplification of this expression gives $C_d = C_o$ as the condition to be satisfied for a zero resultant repulsive stress (R). Physically, this means the complete separation of the particles, with bulk-liquid solution concentration being equal to the mid-point solution concentration. The effect of this idealization, together with the assumptions made in formulating the double layer theory, are evident in Figure 6.8.

The basic assumption made in developing the theoretical curve shown in Figure 6.8 is that the repulsive stress within the sedimented soil to be equal to zero. This assumption holds good both for a fully dispersed particle suspension (as shown above) and a flocculated sediment. Experimental evidence to this effect was found, when bentonite/kaolinite (1:2 mixtures) were sedimented at room temperature (25°C) and 0°C. As noted in chapter 5, temperature had no effect on the equilibrium sediment volume

As indicated by Equation 3.11, repulsive stress is directly proportional to the absolute temperature (273+°C). Sensitivity of repulsive stress, R, to temperature was evident in synaerisis consolidation testing, where it was found necessary to perform these experiments in a temperature controlled room. Since a clay suspension or a flocculated clay sediment (under equilibrium) did not possess a significant repulsive stress, the temperature effect could be ignored.

In a flocculated clay, at equilibrium under the buoyant unit weight of the overlying material, the stress due to the attractive force A is very large, compared to either R or σ_s (Scott, 1965). The assumption, $R \cong 0$, holds good under these circumstances, too.

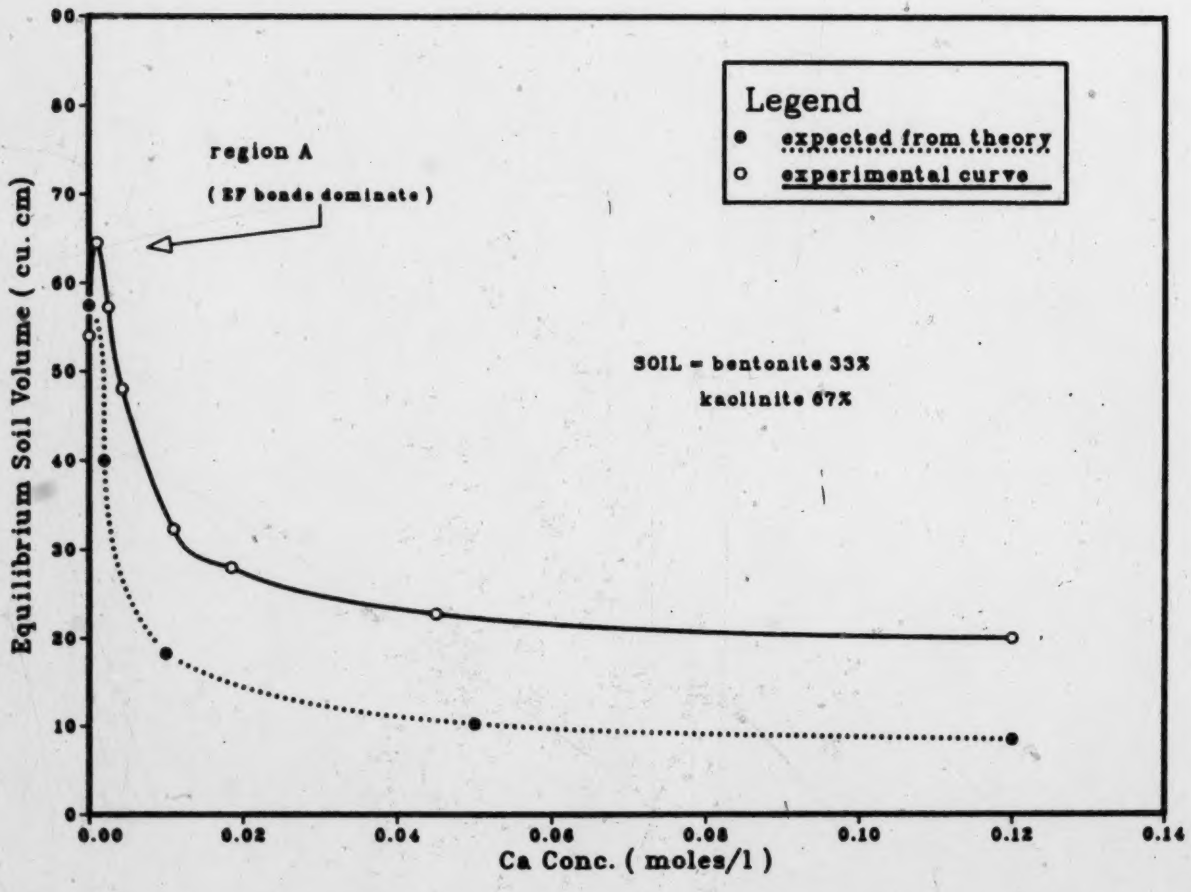

Figure 6.8. Equilibrium Soil Volume vs Calcium

6.3.4 Interpretation of Sedimentation Results

The foregoing discussion managed to establish that the influence of the concentration and kind of cations in the liquid phase, on the interparticle distance and the final sediment volume, could be explained by the theoretical considerations of the electrical double layers. An increased concentration of cations reduces the range of the double layer, thus reducing the repulsive force between the particles (Pusch, 1970).

According to Bolt (1956), parallel arrangement of the particles corresponds to the position of minimum free energy of the system, which means that deviations from this arrangement should be very small. Therefore, the calculation of the interaction between double layers, particularly the repulsive stress, is generally based on the assumption that all particles have parallel basal surfaces, so that the forces induced by the particle edges need not be considered.

In fact, experimental evidence obtained during this research program, supports this theory of interaction of forces. A series of sedimentation experiments performed with pure clay-calcium solution suspensions provides an illustration (Figure 6.8). A direct comparison was made between the actual sedimentation volume (from experiments) and the theoretical sediment volume calculated by using the Gouy-Chapman diffuse double-layer theory and the van't Hoff osmotic equation.

Figure 6.8 provides the opportunity to verify the fundamental explanation of the influence of electrolyte concentration on interparticle forces and clay particle flocculation. Double-layer theory, as adapted for the sedimentation situation, seems to underpredict the equilibrium sediment soil volumes. At low concentrations of electrolyte (except in "region A", which is discussed in detail in the next section), theoretical and experimental values are in closer agreement. Considerable idealizations made during the development of theory and in adaptation to the sedimentation situation should explain the quantitative disagreements. Several factors may contribute to the observed differences. They include;

- incapability of the double-layer theory to account for the existence of non-parallel particle orientations,
- presence of clay particles as tactoids, rather than individual particles,
- presence of "dead volumes".

As noted in Section 6.2, at lower void ratios, the increases in repulsive stress caused by non-parallelism may be cancelled out by corresponding increases in attractive stresses. At very high void ratios (at very low Ca^{+2} concentrations, void ratios could be as high as 15-20) a similar situation would not arise but non-parallelism contributes to the "dead volume", as evident in "Region A".

In fact, the classical double-layer theories give a much too simplified picture of interacting soil particles.

Flocculation of soil particles are known to create various types of particle arrangements (see Appendix C). The forces prevalent within the soil-electrolyte system control the particle arrangement, which in turn dictate the magnitude of the equilibrium sediment volume.

Situation in "Region A"

Behaviour of a clay soil suspension at very low cation concentration is not directly related to the understanding of clay leachate interactions as applicable to compacted clay liners. However, a study of such behaviour would shed more light on the characteristics of the interparticle forces existing within a clay mass. The following discussion is directed at discerning the fundamental aspects of the interparticle forces, and broadening the knowledge of clay suspensions as a whole. Such a knowledge is clearly needed for the interpretation of Figures 5.18, 5.19 and 6.8.

The flat unit-layer surfaces are not the only surfaces of the plate-like clay minerals; they also expose an edge surface area. This is much more evident with kaolinite particles, where 12 percent of the total area consists of edges, as opposed to only 2 percent for montmorillonite. There is a strong possibility that, in a neutral clay suspension, a positive double-layer is created on the edge surfaces owing to the expanded aluminum sheet (van Olphen, 1963).

At the edges of the plates, the tetrahedral silicon sheet and the octahedral alumina sheet (see appendix B for an explanation of the clay mineral structure) are disrupted, and primary bonds are broken. The part of the edge surfaces at which octahedral sheets are broken, will carry positive double layers in acid and neutral solutions and negative double layers in alkaline solutions (Pusch, 1970). The anion adsorption capacity of clays (although small) is thought to be the result of the positive edge charge (Umehara and Gillman, 1981).

The occurrence of the positive double-layer could explain the phenomenon observed in the "region A". In deionized water (with very low cation concentrations being supplied by the soluble salts in bentonite and kaolinite), both double layers on the clay particle are well developed. Internal mutual flocculations are encourged by EF (edge to face) associations. At this point, EF attraction outweighs FF (face to face) repulsion and an increase in viscosity is indicated (Nickel, 1977). The particle arrangement observed here approaches the "non-salt flocculation" fabric proposed by Lambe (1958a).

When the Ca^{+2} concentration is increased to about 0.002M, both negative and positive double layers are compressed. The addition of a small amount of electrolyte results in dispersion, because of reduction of the positive edge charge by anion adsorption. Increasing the salt content causes flocculation, with a parallelic particle arrangement as a result of decreasing extensions of the double layers. The domination of FF repulsion causes the particles to become disengaged and dispersed, creating a higher equilibrium sediment volume.

Since the classical double-layer theory is only applied to the interaction between negative double layers (negating the existence of a positive double layer), the theory, as adapted here, fails to account for the phenomenon observed in region A.

The peculiar behavior of clay suspensions at very low electrolyte concentrations is of little practical interest. However, it shows the importance of edge charges in determining the fabric of clay minerals. Presence of kaolin, with a larger edge area, contributes tremendously to the existence of EF bonds within a soil system. Positively charged edges of kaolin would associate with negatively charged faces of the montmorillonite particles. This can occur even at higher Ca^{+2} concentration and partly explain the occurrence of large "dead volumes" within the sedimented soil mass.

Effect of Tactoid Formation and "Dead Volumes"

When the Ca^{+2} concentration is increased beyond 0.005M, the decrease in double-layer thickness shows a greater effect on FF repulsion. When the soil is sedimented in the high concentration Ca^{+2} solution, the development of the negative double layers are restricted. The particles are able to come closer to each other.

Under such conditions soils flocculate, with a higher degree of particle parallelism than in the low electrolyte solution. The individual plates form an open stepped arrangement of EE, FF and EF contacts known as a "salt flocculated structure" (Appendix C). Salt flocculated, or the "bookhouse" structure (Sides and Barden, 1971), would contain smaller domains or tactoids, often consisting of a few particles arranged parallel to each other (as a packet or a book). The overall arrangement is a random or open structure, with large macrovoids in between these domains.

However, the theory assumed individual particles to be distributed in a homogeneous parallel arrangement, and an equal effective, half spacing, d, between all the particles.

By equating C_d to the bulk liquid Ca^{+2} concentration (Equation 6.8) it was possible to calculate the effective half spacing between each pair of interacting clay particles. Equation 6.3 was used for this purpose. Thereafter, assuming homogeneous fabric and uniform distribution of the long range interaction over the soil particle, the following equation was used to calculate the void ratio (and the water content) and then the equilibrium sediment volume:

$$e = G_s \gamma_w S d \tag{3.12}$$

Na montmorillonites, in very dilute solutions, are not known to exist as tactoids or clay particle packets containing several clay layers. The tactoids of sodium-saturated montmorillonite can be dispersed into individual layers at low electrolyte content. A particle of such a dispersed montmorillonite is only one layer, or 10 Angstroms thick (Yong and Warkentin, 1975). The double-layer theory holds good at such a situation, as evidenced by the experimental results. However, calcium montmorillonites are known to form tactoids in water. When Na montmorillonite particles are dispersed in high strength Ca^{+2} solutions, Na^{+} in the exchangeable complex would be replaced by Ca^{+2}. The presence of Ca^{+2} encourages tactoid formation, and the formation of a fabric peculiar to calcium montmorillonite. Aylmore and Quirk (1959), noting the presence of an internal and external surface (only the external surface being available for particle interaction), proposed the following model for the fabric of calcium montmorillonite.

The clay mass could be viewed as a porous medium, consisting of a clay matrix and a distribution of relatively large voids. Within the clay matrix, the particles are in parallel orientation over regions of approximately 100 Angstrom diameter. These regions of parallel orientation are called "tactoids" or domains (Aylmore and Quirk, 1959,1962).

The clay matrix consists of many tactoids or domains in more or less a random orientation (Chattophadhyay, 1972).

The large pore model proposed by Aylmore and Quirk (1959) is essentially a honeycomb fabric of a clay matrix of particles and water, with large pores also containing water. The water in the large pores is not associated with double layers and must be considered "dead" and subtracted from the water content of the sample, before applying the double-layer theory to the remaining water (Warkentin _et al._, 1957).

The non-parallel plate model has essentially the same effect, in which much of the water between the plates could be considered "dead", since the stress transmitted in the region where particles are far apart is very small (Mitchell, 1976).

Bailey (1965) showed that the effect of taking into account the domain formation is to increase the effective specific surface S, thereby decreasing calculated half spacing d. This eventually predicts a lower sedimented volume. The effect of considering the concept of non-homogeneous soil fabric (contributed by both large pore model and non-parallel plate models) is to increase the calculated d spacing and subsequently increase the sedimented soil volume.

Shainberg _et al._ (1971) suggested that the number of particles per plate group to be in the range of 4 to 10, lower values being associated with the low pressures.

Blackmore and Miller (1961) related the number of units of crystal sheets per packet to the pressure, to which the sample was subjected to. The number of particles ranged from 4, for a 1/2 kPa pressure, to a highest value of 8. According to these observations, it would seem possible that the sediment clay domains should contain a maximum of 4 particles per domain.

Bailey (1965) proposed a method to account for the "dead volume" in the large pore model. Assuming 4 particles per tactoid, the domain thickness was calculated, as 4 particles at 10 Angstroms plus 3 water layers at 9 Angstroms, to be 67 Angstroms. The total volume of a "unit cell" would then be proportional to the thickness of half a domain plus one domain half-spacing or (33.5 + d) Angstroms. Consequently, the dead volume as a percent of total volume, V_D, will be:

$$V_D = \frac{100\ d_D}{33.5 + d} \qquad 6.12$$

where,

d_D is the portion of d assigned to dead volume.

The presence of clay particles as tactoids and the dead volume concept is taken into account in the calculation of $(R\text{-}A)_3$, for leachate treated soil samples. A fairly good estimate of R_3 was needed to analyze the synaerisis consolidation results using the void ratio-log pressure plot. In the case of interpreting sedimentation results, the existence of a dead volume explains the underprediction of experimental results by classical double-layer theories. A physical examination of the equilibrium sedimented volumes (at high Ca^{+2} concentrations) revealed the existence of such areas within a soil mass (the sediment tends to flow if disturbed, unrestricted by any internal forces).

Based on the experimental evidence and theoretical considerations, it can be concluded that the sediments in high Ca^{+2} concentrations, for the most part, do not behave as a homogeneous soil mass with forces interlocking each particle, but exhibit the presence of tactoids and dead volumes where no interparticle forces extend.

6.3.4.1 FSDP and System Variables

The experimental results, generated during this research program, enabled the elucidation of the relationship between observed sedimentation volumes and the system variables. The detailed discussion (Chapter 5) revealed the following:

- the significant influence of percentage bentonite (%B) and Ca^{+2} concentration on the soil-leachate parameter, FSDP,
- the marginal influence of acetic acid and Na^{+} concentration on FSDP,
- similarities and dissimilarities of the effect of divalent cations, Ca^{+2} and Mg^{+2}, on FSDP,
- peculiar behavior of K^{+} ions (compared with other monovalent cations).

In addition, it was found that all these chemicals, except acetic acid, cause reduction of sediment volumes primarily by changing the characteristics of interparticle forces. The chemical that prevents the development of a double layer to its full capacity generally produces a smaller sediment volume and a higher FSDP. This indicates the capacity of the chemical to cause synaerisis shrinkage in the compacted soil. Higher Ca^{+2} ion concentration, in a sedimenting liquid, allowed only a small sediment volume and in a leaching solution caused a larger strain. Whereas, the effect of Na^{+}, in both cases, was comparatively less.

The theoretical considerations used in interpreting the results generated by sedimentation and synaerisis consolidation processes, strongly suggest that both processes are governed by the same mechanism; ie. changes in interparticle forces caused by the changing electrolyte environment. The presence of a strong correlation (Equation 5.3) between the parameters, FSDP and percent strain, as expected from theoretical considerations, provide confirmation of the existence of a common process among sedimentation and synaerisis shrinkage.

The experimental evidence and the theoretical considerations point to the following general facts regarding the effect of system variables on both FSDP and synaerisis percent strain:

- soils containing higher percentages of high swelling bentonite exhibit a higher FSDP and a higher percent strain,

- divalent cations (in this case Ca^{+2}), produce a higher response than monovalent cations (Na^{+}),
- in general, higher cation concentrations produce higher responses (magnitude depends largely on the swelling potential of the clay soil).

6.4 EFFECT OF LEACHATES ON HYDRAULIC CONDUCTIVITY AND COMPRESSIBILITY

6.4.1 Hydraulic Conductivity

Ridley (1985) identified several characteristics of highly concentrated, electrolytes which he proposed to be capable of altering the permeability of a compacted clay liner. His observations were based on permeability studies performed with brine on compacted clay soils.

The characteristics are:

- flocculating potential that the high electrolyte leachate will have on the soil,
- the potential to cause interlayer contraction in montmorillonite clay mineral,
- the potential to alter the pore-size distribution,
- the dissolution of certain mineral fractions.

Although highly concentrated MSW leachate, as well as brine, is capable of flocculating a dispersed clay suspension or producing a flocculated soil structure or both (see appendix C for clay microstructure explanations), such a process could not possibly occur within a compacted clay soil. A dispersed soil structure (parallel particle arrangement) represents the lowest potential energy condition for a soil system and very high energies are required to change a dispersed system to a flocculated condition. However a high strength leachate causing flocculation in a clay dispersion by reducing the net interparticle repulsion between clay particles, is capable of altering the pore-size distribution in the clay soil. Therefore, rather than being the cause of permeability increases, the enhanced flocculating potential become another manifestation of the change in interparticle physico-chemical forces, caused by electrolyte-clay interaction.

The result of interlayer contraction, which is caused by the reduction of repulsive stress, would be to alter the pore-size distribution, specifically the increase of macropores at the expense of a large number of micropores.

Ridley (1985) concluded that stripping of amorphous material and dissolution of mineral fractions during permeation causes only a marginal increase in permeability. This would leave us with the alteration of pore size distribution and an increase in effective porosity, as the major cause for any observation of hydraulic conductivity increases across compacted soil material, resulting from exposure to brine.

The leachate used in series #1 experiments was capable of diminishing the net repulsive stress as proven from shrinkage consolidation results. Changes in the pore size distribution would be expected, but a reduction in hydraulic conductivity values was not observed in the results presented in Table 5.15.

If the axial stress applied to the soil was kept at a minimum, which is the case when a standard permeability test is performed (as described by Olson and Daniel, 1981), and/or the volume shrinkage caused by clay-leachate interaction was prevented, there would definitely be a change in pore size distribution as a result of leachate permeation. However, in the case of the synaerisis-shrinkage experiments, a moderate axial stress of 80 kPa was maintained throughout the leaching process and volume shrinkage was allowed. This may effectively prevent the formation of a large number of macropores.

The hydraulic conductivities presented in Table 5.15 were calculated by fitting Terzaghi's theory of consolidation to the laboratory time settlement observations and extracting hydraulic conductivity, k, from the calculated coefficient of consolidation, C_v (Taylor, 1948). The fitting operation utilized the "square-root" fitting method, in which fitting is performed at an average 90% degree of consolidation.

A question may be raised as to how well the k values calculated with this method, compare with values measured by direct permeability testing. Mesri and Olson (1971a), reported such comparisons were made by Terzaghi himself at the time of theory development and found satisfactory agreement. Later, Taylor (1942) presented comparisons for remolded samples of Boston blue clay, based on the square root fitting method, and showed that the measured coefficients of permeability generally exceeded the calculated values. Olson and Daniel (1970) agreed with

Taylor's contention that this underprediction of measured values was caused by the Terzaghi's assumption that total resistance to compression was provided by the soil permeability alone.

The delay in compression, or the time dependancy of consolidation settlement, is not caused merely by the time required for the water to be squeezed out (or hydraulic conductivity), but partly by hydraulic conductivity and the structural resistance of the clay itself. Mesri and Olson (1971a) reported their observations on the possible comparison of calculated and measured hydraulic conductivity values and concluded that the calculated k values underpredict measured values by only about 5-20 percent, provided that the clay is normally consolidated at the time of k determination.

The k values, generated by the consolidation during this research, generally are in agreement with the published values for montmorillonite, kaolinite mixtures (Mesri and Olson, 1971a, Mitchell, 1976). However, caution should be excercised in the interpretation of leachate effect on hydraulic conductivity. As seen from the above discussion, the normally consolidated soil samples used in all sets of experiments would show k values in close agreements with measured values. However, the leachate permeated soil seem to behave quite differently from the non-treated soil. This is evident from the increased compression indices observed (see Section 6.2) and higher compressibility indicated by the coefficient of consolidation, C_v (Table 6.3), and compressibility, m_v (will be discussed in section 6.4.2). The difference in the magnitude of structural resistance offered, before and after leachate permeation, may also contribute to the calculated k values. Attempts were not made to isolate (or calculate) the contribution made by this factor. A better understanding of the leachate effect on the soil parameter k is expected, once the long term permeabilities are available and a comparison between calculated and measured values (especially after leachate permeation) are made.

A given soil may exhibit many fold variations in hydraulic conductivity, k, as a result of changes in the fabric, void ratio and water content (Mitchell, 1976). Application of pressure leads to an increasingly tortuous flow path in the direction of the applied pressure. In addition, increasing consolidation pressure decreases the total void space available for liquid migration. These observations have important implications in interpreting the observed k results. Since a decrease in void ratio decreases the calculated k values, only k values at the same void ratio (or consolidation pressure for a given soil) are directly comparable. Rather than comparing pre- and post-leachate k values, comparison of post-leachate k in a given soil at the same void ratio (or consolidation pressure) should provide more information about the ability of a liquid to change the basic soil parameter k. Series #3

TABLE 6.3

POST-LEACHATE CONSOLIDATION COEFFICIENTS

CELL #	LEACHATE Ca conc. + Base	COEFFICIENT OF CONSOLIDATION AFTER LEACHATE TREATMENT $(cm^2/s) \times 10^{-3}$	CHANGE IN C_v CAUSED BY LEACHATE $(cm^2/s) \times 10^{-3}$
C-3.2	Base Only	1.93	2.57
C-3.3	0.002 + Base	1.66	2.25
C-3.4	0.004 + Base	1.89	2.66
C-3.5	0.009 + Base	1.86	2.64
C-3.6	0.016 + Base	1.76	2.59
C-3.7	0.022 + Base	2.46	3.41
C-3.8	0.045 + Base	2.75	3.73
C-3.9	0.090 + Base	3.66	4.68

Base liquid consists of : - 0.017 moles/l, Na
- 250 mg/l, HAc

results, presented in Chapter 5, provide this opportunity. The post-leachate k values are extracted from Table 5.6 and shown in Table 6.4, along with the initial and final void ratios for the axial pressure 160 kPa. The results strongly indicate an increasing trend for the k value with increasing calcium ion concentration.

As noted from these results, large increases of hydraulic conductivity, k, are not anticipated across a compacted clay liner. An increase in the soil microparameter, k, is expected when the size, shape and the geometrical arrangement of the pore spaces are changed from a less to a more conducive state for liquid flow. However, the overall effect of such a change may not be several orders of magnitude but less than 100% in most of the cases. This is evident from the porosity-hydraulic conductivity equation presented earlier in chapter 3. The simplified version of Kozeny-Carmen relationship, as proposed by Taylor (1948), clearly indicates that hydraulic conductivity is directly proportional to the square of effective particle diameter (d_e). The effective diameter could be reduced as a result of the diminution of the double layer thickness. The resulting increases, due to this effect, in the hydraulic conductivity of six clay soils in Canada, when tested with brine, were found to be in the range of 0.1 to 0.2 orders of magnitude (Ridley, 1985). This small increase in k is not of major concern to the engineers and is negligible compared to the possible large increases (several orders of magnitude)

TABLE 6.4

POST - LEACHATE k VALUES - SERIES # 3

CELL #	VOID RATIO		$k \times 10^{-10}$ (m/s)	Ca Concentration (moles/l)
	PRE LEACHATE	POST LEACHATE		
C-3.2	2.94	2.91	1.87	0
C-3.3	2.80	2.75	1.61	0.002
C-3.4	2.78	2.73	1.97	0.004
C-3.5	2.85	2.73	1.95	0.009
C-3.6	2.86	2.57	2.12	0.016
C-3.7	2.79	2.37	2.60	0.022
C-3.8	2.84	2.32	3.15	0.045
C-3.9	2.83	2.24	3.74	0.090

in bulk permeability caused by volume shrinkage and subsequent cracking.

6.4.2 Soil Compressibility

The phenomenon of consolidation is one which depends upon time, soil and pore fluid properties, and axial pressure application. Compressibility describes the characteristics of compression behavior not solely covered by consolidation and includes the general performance of volume change (with or without shear distortion) under load.

The coefficient of consolidation, as defined by Terzaghi, is related to the hydraulic conductivity, k, and modulus of volume change, m_v, by

$$C_v = k \left\{\frac{1}{m_v \gamma_w}\right\} \tag{6.13}$$

As the void ratio of a soil mass decreases, both k and m_v decrease rapidly; however, C_v, which depends on the ratio (k/m_v) remains fairly constant within a considerable range of pressure (Singh, 1976). If clay-leachate interactions would cause major microstructural changes, it should be evident in the corresponding C_v values. However, no such indications are shown in the results presented for Series #1 cells.

No major changes in microstructure are evident from the comparison of C_v values before and after leachate permeation. Results from microstructural investigations, given in Section 5.5, also supported this observation. The micrographs, obtained after leachate permeation, compared well with the non-leachate treated soil; they did not deviate very much from the more dispersed structure shown.

However, as evident from the Figures 5.1 and 5.2, soil treated with high Ca^{+2} did not consolidate very much on the 160 kPa load increment. In fact, the total consolidation of the soil C-1.2, which was treated with 0.017M Ca^{+2}, was almost twice the settlement shown by soil C-1.5, treated with 0.045M Ca^{+2}. This indicates the stabilization of the soil under leachate permeation.

Soil stabilization definitely has strong implications in designing landfill liners for zero or very little volume shrinkage, and was studied in series #8 experiments (which will be discussed in the next section).

Results obtained from series #3, as presented in Table 6.3, clearly indicates the effect of cations on coefficient of consolidation, C_v. The diminution of double layers by increased cation concentration causes increased rate of water flow and increased rate of consolidation, on further effective stress change. Almost 125% increase in C_v was observed as a result of 0.09 M Ca^{+2} permeation. The reduction in double layers by cations seem to help in stabilizing the soil; viz, increased resistance to further volume change due to changes in true effective stress.

6.5 DIRECT PERMEABILITY, LONG TERM CONSOLIDATION AND SOIL STABILIZATION EXPERIMENTS

6.5.1 Direct Permeability Tests with MSW Leachate and NaCl Brine as Permeant

The hydraulic conductivity values, calculated by fitting Terzaghi's theory of consolidation to the time settlement curves, were at least twenty percent lower than the actual values (Mesri and Olson, 1971b). Furthermore, because of the dependency of hydraulic conductivity on void ratio and bulk density, a direct comparison of pre-leachate and post-leachate hydraulic conductivities could not be performed. Any increase caused by leachate treatment is masked by the decrease in void ratio caused by the load increment (a decrease in void ratio reduces the magnitude of hydraulic conductivity). However, time settlement results obtained at similar void ratios, on different leachate (varying Ca^{+2} concentration) treated soils, clearly showed the effect of cations on hydraulic conductivity, k. Direct permeability tests were performed to confirm these results. Most of the measurements were taken at low hydraulic gradients.

Two sets of permeability tests were performed with "standard leachate" and 30% NaCl brine as the permeants. In Table 6.5, results pertaining to the direct permeability test performed on soil A modified with 20% bentonite, are presented. These results were obtained by performing direct

TABLE 6.5

HYDRAULIC CONDUCTIVITY VALUES BEFORE AND AFTER LEACHATE PERMEATION

(Test Conditions: Surcharge Pressure = 80 kPa, Leachate, 160 kPa
Soil = Soil A + 20 % bentonite
Liquid = standard leachate
Hydraulic Head = 5 and 55)

Point of Permeability Measurement	$k \times 10^{-10}$ (m/s)	Comments
Before leachate addition	2.3	Average over 3 days
# of days after leachate addition		
10 d	1.5	Contribution from synaerisis shrinkage
15 d	1.4	Small contribution
17 d	1.3	No contribution from shrinkage
22 d	1.6	"
29 d	1.4	"
38 d	1.5	"
50 d	1.5	"
7 d after 160 kPa load increment	0.7	

permeability experiments with deionized water and standard leachate, both at low and moderate (5 and 30) hydraulic gradients. Permeability test results obtained with standard leachate, several days after leachate charging, showed a slight decrease in k values.

Permeability measurements during synaerisis consolidation was avoided. However, there was some contribution to the 10 d (post-leachate), k value from synaerisis consolidation, because a 0.0025 mm/h shrinkage rate was observed during this measurement. Permeability and deflections were monitored for a period of 50 days after initial addition of liquid. After early fluctuations, the k values held steady at about 65% of the value obtained with deionized water at an effective stress of 80 kPa.

Reduction of pore spaces, during synaerisis shrinkage, contributed to the k value decrease. The effect of void ratio reduction on k was evident in the measurement obtained 7 days after increasing the stress to 160 kPa (on the leachate treated soil), which registered a k value of 0.7 x 10^{-10} m/s.

As evident from these results, it is obvious that a possible increase in k by leachate charging is masked by the decrease in k caused by closing pores. The standard leachate, during permeation, could cause two effects. First, by reducing bound water associated with clay particles, leachate tends to change the pore size distribution, increasing the number of larger pores. If the void ratio was kept constant, this would result in an increase in k. Second, by increasing effective stresses within the soil matrix, leachate causes the soil mass to consolidate, effectively diminishing the pore sizes. Depending on the surcharge pressure acting on the system, a net increase or decrease of k would occur. In this case, the net result was a slight decrease in k. Similar results were obtained in the replicate sample (results presented in Table 6.6). These direct permeability results confirm the calculated k values obtained from consolidation results (presented earlier), and validate the notion mentioned earlier that calculated k values are good first approximations of clay liner hydraulic conductivity values.

6.5.1.1 Brine Permeability

Two soil samples, a bentonite/kaolin 1 to 4 mixture and a 2 to 3 mixture, were permeated with the 30% NaCl brine. Corresponding permeability results are tabulated in Tables 6.7 and 6.8. As the reservoir liquid was changed to brine a sudden increase in k was noted for both soil samples. The sudden increase in drainage was accompanied by rapid consolidation of samples, which caused pore water to drain out from the soil system and appear as permeating liquid in the U-tube manometer. The majority of the liquid appearing in the permeability measuring system can be shown to arise from this mechanism

TABLE 6.6

HYDRAULIC CONDUCTIVITY VALUES BEFORE AND AFTER LEACHATE PERMEATION

(Test Conditions: Surcharge Pressure = 80 kPa, Leachate, 160 kPa
Soil = Soil A + 20 % bentonite
Liquid = standard leachate
Hydraulic Gradient = 5 and 55)

Point of Permeability Measurement	k x 10^{-10} (m/s)	Comments
Before leachate addition	2.4	Average over 3 days
# of days after leachate addition		
15 d	1.6	Very little or no contribution from consolidation
21 d	1.5	"
28 d	1.5	"
33 d	2.1	"
36 d	1.7	"
# of days after 160 kPa load increment		
8 d	0.9	
11 d	0.6	
13 d	0.7	
15 d	0.6	

TABLE 6.7

LOW GRADIENT DIRECT PERMEABILITY TESTS - EFFECT OF BRINE ON HYDRAULIC CONDUCTIVITY (k)

(Test Conditions: Surcharge Pressure = 80 kPa, Leachate, 160 kPa
Soil = bentonite 20 % & kaolin
Liquid = 30 % NaCl brine
Hydraulic Gradient = 5 to 7)

Point of Permeability Measurement	$k \times 10^{-10}$ (m/s)	Comments
8 d before brine addition	2.3	
6 d "	1.2	
1 d "	1.2	
# of hours or days after brine addition		
8 h	10.0	Large contribution* from shrinkage rate = 0.07 mm/h
16 h	18.0	Shrinkage rate = 0.05 mm/h
32 h	15.0	" = 0.03 mm/h
68 h	2.2	" = 0.004 mm/h
82 h	0.8	" = 0.003 mm/h
after 4 d	no flow	No deflection

*NOTE: 1 mm deflection contribute 1.57 mL liquid to the drainage (assuming equal double drainage)

TABLE 6.8

LOW GRADIENT DIRECT PERMEABILITY TESTS - EFFECT OF BRINE ON HYDRAULIC CONDUCTIVITY (k)

(Test Conditions: Surcharge Pressure = 80 kPa, Leachate
Soil = bentonite 40 % & kaolin
Liquid = 30 % NaCl brine
Hydraulic Gradient = 6 to 9)

Point of Permeability Measurement	$k \times 10^{-10}$ (m/s)	Comments
8 d before brine addition	0.09	Very low permeability
6 d "	0.10	"
1 d "	0.07	"
# of hours or days after brine addition		
8 h	7.80	Large contribution from shrinkage rate = 0.09 mm/h
32 h	23.00	" = 0.06 mm/h
68 h	3.20	" = 0.01 mm/h
82 h	1.10	" = 0.006 mm/h
more than 4 d	no flow	No deflection

For an example, if equal double drainage is assumed, at 32 h (in Table 6.7), 72% of the liquid collected in the U-tube manometer could be attributed to pore water squeezed out during consolidation. Elimination of the consolidation contribution would reduce the k value from 23 x 10^{-10} m/s to 6.5 x 10^{-10} m/s, which still is almost two orders of magnitude higher than the deionized k value. Brine, moving rapidly across the soil sample causes a rapid change in interparticle forces and also a change in the pore size distribution. Creation of larger pores allows brine to move faster across the soil sample. Resulting higher permeation and subsequent consolidation produces the increased flow.

However, consolidation caused by effective stress changes, causes a reduction in pore sizes and a reduction in hydraulic conductivity. This is evident in both samples. Beyond 32 h, permeability kept decreasing until 4d. No further flow or shrinkage was observed after 4 days, in both samples. At this point, all pore spaces were occupied by the high concentration brine, which essentially plugged the pores.

Blocking of pore spaces by salt could be explained as follows: Assuming hydration numbers of 4 and 8 for Cl^- and Na^+ respectively (Glasstone, 1978), it is possible to calculate the amount of free water in a NaCl solution. As 3.69 g of H_2O is needed to fully hydrate 1 g of NaCl, beyond 27% NaCl, no free water exists in a NaCl solution (NOTE - with the solubility of NaCl at room remperature being 36%, exceptions to this rule are indicated). Therefore, once the pore water was replaced by brine, no free water can exist within the soil system.

This phenomenon became an important factor mainly because of the low gradients used in this experiment. Since, the permeability tests were performed on the consolidation apparatus, without provisions for increasing the hydraulic head, applied hydraulic gradients ranged between 6 and 9.

However, in practice (in brine ponds), higher hydraulic gradients are not expected. As indicated by these results, in such situations, a decrease in hydraulic conductivity, rather than an increase is foreseen. Therefore, the increases in field bulk permeabilities (in brine ponds) could only be explained by the development of cracks within the clay liners. The rapid shrinkages observed during these experiments definitely reinforces this hypothesis. However, further research may be needed to verify these findings and draw definite conclusions.

6.5.2 Synaerisis Consolidation Experiments at High Pressures and with Brine as the Permeant

6.5.2.1 High Pressure Consolidation Experiments

The decision to perform synaerisis consolidation experiments at 80 kPa external vertical stress had a major disadvantage. Because of the adopted artificial sedimentation technique for sample preparation, samples could not be tested at very low void ratio values. The field clay liners are expected to show a void ratio value of about 1.0. At 80 kPa, most of the soil samples showed void ratio values ranging from 2.0 to 4.5. Since the volume shrinkages observed were for higher initial void ratios, it was essential to find out whether these results could be extrapolated to field conditions.

Two samples containing 40% bentonite and kaolinite prepared at 560 kPa, consolidated 1.142 mm and 0.981 mm, when permeated with the standard leachate. The final void ratios were 1.56 and 1.54. The duplicate trials, produced an average percent strain value of 6.5, which was lower than the shrinkages produced in trial # C-3.8. Trial # C-3.8 performed under 80 kPa pressure but with the same clay mineral combination and leachate, produced a synaerisis shrinkage of 3.65 mm and a corresponding percent strain of 13.7.

Consolidation of the highly expanding soil to a lower void ratio, using a higher consolidation pressure, creates a larger repulsive stress among clay particles (or a larger swelling pressure within the soil matrix). However, higher van der Waals attractive stresses, caused by the close proximity of the clay particles (attractive stress being inversely proportional to the third power of d), tend to reduce the net (R-A) interparticle stress. Although negligible at higher void ratios, contribution from the attractive stress to the (R-A) component of the true effective stress (σ^*) may become significant at low void ratios.

The standard leachate, by reducing the double layers, is capable of releasing the swelling pressure in the soil. Subsequent increase in σ^* causes the observed synaerisis consolidation. The observed average percent strain (at low void ratio) being less than 50% of percent strain at the higher void ratio (initial void ratio for cell C-3.8 was, 2.84), shows the role played by attractive stress, A, and difficulty of causing shrinkage in a soil compacted to a low void ratio. In the case of soils consolidated to high pressures or low void ratios, larger effective stress changes are needed to cause any volume shrinkage.

Therefore, to observe a similar synaerisis strain as in the case with cell C-3.8, a very large effective stress change was necessary. The magnitude of stress change needed could be fathomed by looking at the stress-strain behavior of the sample before leachate addition. Consolidation strain caused by the stress increment, 560 kPa, in both soil samples was 17.4%. As seen from the virgin curve in Figure 6.9, at least 400 kPa effective stress change is necessary to cause the 7.05% strain observed for cell #1.

The necessity of high effective stress changes to cause shrinkage and the possible significant influence of attractive stresses, tend to limit the magnitude of synaerisis shrinkage caused by high electrolyte leachates. However, as long as repulsive stresses are available within the soil system, some amount of synaerisis shrinkage should be expected.

Furthermore, as indicated by experimental results, the rate of synaerisis shrinkage tends to reduce with decreasing void ratio, which is a direct result of retardation of flow across the soil sample. The influence of rate of shrinkage on crack formation is not quite clear at the moment. A study on formation and behavior of synaerisis cracks in a landfill environemnt should take this factor into consideration.

6.5.2.2 Synaerisis Consolidation Caused by 30% NaCl Brine

While performing direct permeability tests on 20% bentonite/80% kaolin and 40% bentonite/60% kaolin soil samples, it was possible to monitor the synaerisis shrinkage caused by 30% NaCl brine. In addition to providing information on the volume change behavior of soils under the influence of brine, these data provided

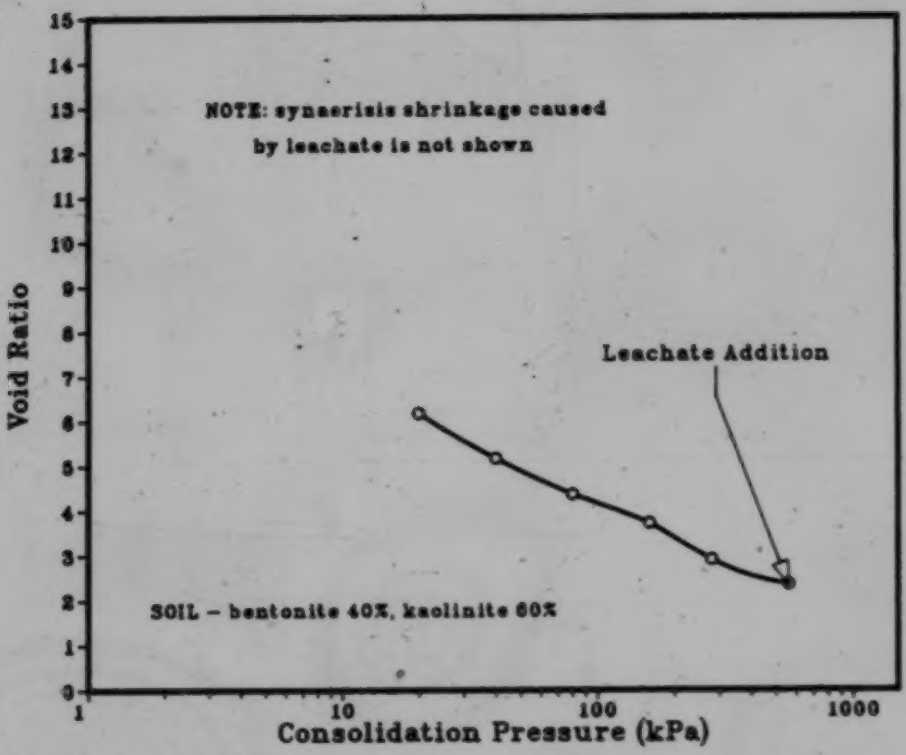

Figure 6.9 Void Ratio – Log Pressure for High Pressures

valuable data which could be used in explaining the abnormal, sudden increase in k due to brine treatment.

Unlike with MSW leachate, 95% of the total volume shrinkage occurred within the first 72 hours of brine permeation. The rapid migration of NaCl across the thin soil samples (pre-brine thicknesses are 28.5 mm and 26.2 mm for high and low bentonite samples, respectively, caused total shrinkages of 2.48 mm (percent strain = 8.7) and 4.29 mm (percent strain = 16.4), in bentonite = 20% and bentonite = 40% soil samples, respectively. The synaerisis shrinkage was essentially complete within 140 hours.

The total synaerisis shrinkage was higher than that observed for MSW leachate. However, the difference was less than 20%. For example, 40% bentonite and 60% kaolin sample treated with standard leachate registered a total percent shrinkage strain of 13.7%, which is 83.5% of the shrinkage produced by brine. Therefore, the major difference between the two permeants seem to be the very high rate of shrinkage observed with 30% NaCl brine.

6.5.3 Effect of Variable Leachate Composition and Soil Stabilization on Volume Shrinkage

6.5.3.1 Effect of Leachate Composition Variations

The composition of the landfill leachate depends, among other variables, on the age of the landfill. As noted in chapter two, the leachate collected at the bottom of the landfill in the early stages of its operation would be the liquid pressed out from the solid waste itself. The cation concentration in this leachate is low. As the decomposition proceeds, more and more cations are added to the leachate increasing its strength. Therefore, one trial (cell C-8.4) was performed to simulate this situation in the laboratory. The resulting time-settlement curve is shown in Figure 5.9.

First the soil (Bentonite 70% and Kaolinite 30%), was charged with the "standard leachate". After 10 days, the liquid was changed to a solution containing 0.09M Ca ions. This liquid change caused an increase in the settlement rate. The soil sample was permeated with this liquid for the next 26 days. Thirty three days after changing the reservoir liquid to a Ca solution, the soil sample had settled a total of about 5.4 mm. The majority of this shrinkage was observed with 0.09M Ca^{+2} solution. At this point, Ca concentration of the leachate was increased to 0.45M, which was 10 times the original Ca^{+2} concentration. This liquid change (Ca^{+2} from 0.09M to 0.45M), did not cause a significant increase in settlement. In fact, the soil sample exhibited an additional settlement of only 0.4 mm.

The total recorded deflection was 5.85 mm (or a percent strain of 23.94). On the other hand, for the same soil with standard leachate, a percent strain of 23.9 was observed in cell C-5.8.

Two important conclusions can be made from these observations. First, the volume change caused by moderate

Ca^{+2} concentration (0.045M) was approximately equal to the shrinkage caused by high Ca^{+2}, 0.45M. This was indicated by the earlier (R-A) calculations (Section 6.2) where it was found that standard leachate was capable of reducing net physico-chemical stress by more than 90%. Therefore further increments in Ca^{+2} concentration could not cause a further reduction in repulsive stress or an increase in σ^*. Therefore the contribution from the increases in Ca^{+2} concentration (beyond Ca^{+2} = 0.045M) to the volume shrinkage would be very small.

Secondly, it would seem appropriate to conclude that the effect of leachate or cations to be additive. Change in slope shown at the point of second liquid change (from Ca^{+2} = 0.045M to Ca^{+2} = 0.09M, in Fig. 5.9), indicated a faster rate of settlement for high concentrated leachate. This was expected because the higher concentration was expected to push through the cations at a faster rate, because of increased diffusion rates. Although the rate of settlement was changed by higher calcium, the final volume shrinkage, which is a function of the initial net particle physico-chemical stress, was not altered by a large amount.

The results obtained during this test run support the validity of applying the effective stress concept to explain the synaerisis shrinkage phenomenon, and also vindicate the use of a "standard leachate" in testing soils for shrinkage. However, it should be remembered that the synaerisis shrinkage is a function of repulsive stress, R, or osmotic potential, and the amount of cations needed to suppress R, and cause shrinkage, is dependent on the magnitude of R. At a particular void ratio, the magnitude of synaerisis shrinkage that a soil can exhibit would be a fixed property of the soil. Fixed amounts of cations are needed to cause the full potential volumetric shrinkage. Therefore, natural soils, with smaller physico-chemical repulsive stresses, would require smaller cation concentrations to shrink to their full potential. The suggestion is that a high concentrated brine is not needed to show a large synaerisis effect, but even a less concentrated MSW leachate is capable of causing the same effect, if the shrinkage potentail exists.

6.5.3.2 Soil Stabilization

Concepts of soil stabilization vary enormously among different fields of application. In soil mechanics, it involves the alteration of the properties of the soil-water-air system to those desired for the particular application. The processes normally considered are compaction, drainage, treatment of the soil with admixtures and thermal treatment (Metcalf, 1964).

Virtually all types of soils can be stabilized for almost any purpose. Highly organic soils, with a large or very active clay content, show extreme volume change characteristics and need to be mixed with additives if selected for constructing a compacted clay liner. Lime ($Ca(OH)_2$) is a popular soil additive, used in soil stabilization for highway construction. In addition, to reducing the potential volume change, lime is capable of increasing the shear strength of the soil.

Cells C-8.1, C-8.2 and C-8.3 were loaded with soil mixtures containing 25% Bentonite and 75% Kaolinite prepared with 1,000 mg/L, 5,000 mg/L and 555 mg/L $CaCl_2$, respectively. These were charged with "standard leachate". It was thought that such an approach would give an indication to the selection of a liquid to be used in preparing the clay liner. The time settlement curve, shown in Figure 6.10, indicates that the largest volume change was observed in cell C-8.3. The volume change in cell C-8.2 has reduced considerably, calculated percent strain being 0.08 as opposed to 5.1 for cell C-8.3. The soil prepared with a 1000 mg/L $CaCl_2$ solution exhibited a percent strain of 2.97 which is about 58% of the total void ratio change observed for cell C-8.1. The 5,000 mg/L $CaCl_2$ solution was able to completely suppress the long range interparticle repulsive stress, R, and managed to diminish the synaerisis shrinkage to a negligible level.

Therefore, it would seem feasible to mix Ca^{+2} ions with the compacting liquid, or blend lime with the selected soil, as a means of stabilizing it. The amount necessary would be much less than the 5000 mg/L used herein, as the repulsive stress would be lower. If the composition of the soil is known, it is possible to estimate the amount of Ca^{+2} ion needed to suppress the repulsive stress (which can be

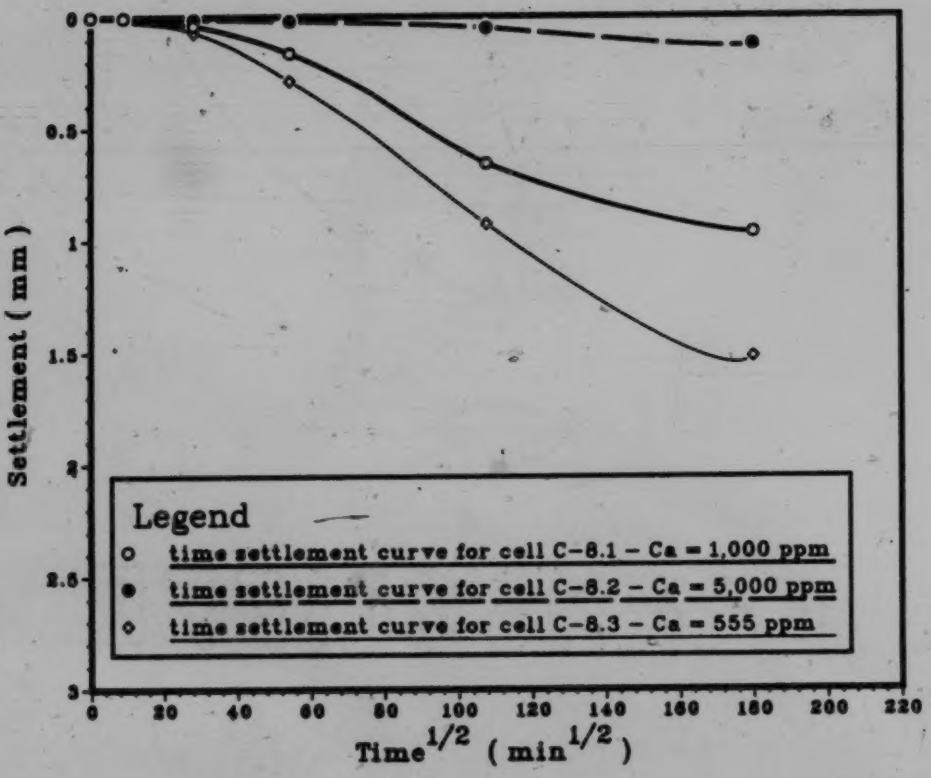

Figure 6.10 Time Settlement Curves for Series #8 cells

calculated using the methods discussed earlier). However as a first approximation, if the exchangeable cation data are known, the Gapon equation can be applied to calculate the concentration of Ca^{+2} ions necessary to displace Na^{+} ions (responsible for the development of the repulsive stress) from the exchange complex.

6.6 IMPORTANCE OF VOLUME SHRINKAGE IN LANDFILL SITUATIONS

By far, most of the municipal solid waste (MSW) today is disposed of in sanitary landfills. This will undoubtedly continue until a cheaper technique is identified. Alternate waste handling technologies, such as recycling, incineration and composting, only result in a reduction of the amount of waste that has to be landfilled (Stegman, 1982 and Maimone, 1985).

As noted in Chapter 2, permeability is currently the primary criterion used in evaluating the suitability of clay liners for containment of leachates. This soil characteristic controls the velocity of fluid flow through the compacted soil and thus the flow volume during a given period of time. This has led many regulatory agencies to adopt a hydraulic conductivity no greater than a fixed value, generally 1 x 10^{-9} m/s.

However, like most other soil properties, hydraulic conductivity is a random variable, and the difficulty with establishing specific limiting values for it is the uncertainty associated with its laboratory determination and the resultant assurance of desired performance.

To overcome these limitations, a variety of approaches ranging from refinement of laboratory hydraulic conductivity measurement techniques (Dunn, 1983) to application of probability theory (Harrop-Williams, 1982), have been attempted. These approaches are mainly directed at eliminating the uncertainty associated with the determination and field application of the soil parameter, k.

However, the approach suggested in this thesis clearly demands the engineer to look beyond k, and concentrate more on the behavior of the clay liner as a whole unit. As argued in earlier chapters, clay leachate interaction, manifested as volume shrinkage, was considered one of the major contributors to permeability increases associated with the landfill situation. The soil parameter, k, could be used as a first approximation. However, legitimacy of performing an expensive, time consuming permeability test to determine this soil parameter is questionable. On the one hand, the calculated k values from the results derived by performing consolidation experiments were comparable with direct permeability test results (at least when changes in k as a result of leachate interaction are being investigated). As well, a consolidation test, if performed following the procedures outlined in this thesis, would provide information on the synaerisis shrinkage behavior of the clay soil.

The information gathered from such a test would allow a design engineer to follow either one of the following pathways:

- make a rational decision on the suitability of the clay soil being tested, and if found necessary, modify the basic engineering properties of the soil, or

- make allowance in the design process to incorporate the bulk permeability changes caused by the inhomogeneities in the clay liner.

If the calculated k values are undesirably high, or large volume shrinkages are observed during the synaerisis consolidation testing, soil should be modified by either adding more clay sized particles or mixing Ca^{+2} ions, as discussed earlier in this chapter. The amount of clay sized particles needed should be determined based upon the hydraulic conductivity of the natural soil. Use of bentonite (or sodium montmorillonite) should be avoided whenever possible. As noted in the experiments with natural soil and bentonite mixtures (cells C-7.4, C-7.5 and C-7.6), even small amounts of Na montmorillonite could cause some amount of volume shrinkage. Kaolin, Ca montmorillonite and even illite (although not tested during the research program, the volume shrinkages associated with this mineral are known to be much smaller than sodium montmorillonite) are capable of reducing the hydraulic conductivity, without increasing the synaerisis shrinkage susceptibility of a natural soil.

The sedimentation testing could play an important role in the soil selection process itself. A large number of competing soils can be tested for potential synaerisis shrinkage, with a range of leachate concentrations, in a very short time. Determination of FSDP values for each soil, with the standard leachate, would provide a good basis for the elimination of problem soils. Whenever the necessity arises for the addition of clay minerals to natural soils, these new mixtures could be tested by the sedimentation technique before proceeding to a long term consolidation test. The sedimentation test would demonstrate the potential increase (if any) in synaerisis shrinkage caused by the addition of clay mineral to the natural soil.

As noted here, the sedimentation experiment could be used as a preliminary screening test in the soil selection process. Calculation of FSDP for the soil would permit prediction of the expected percent strain in a long term synaerisis consolidation test. Although a compacted clay soil is not simply a dense colloidal suspension, the suspensions obtained by performing a sedimentation experiment could offer a physically meaningful point of departure for the study of clay soil-leachate interaction. An in-depth study of mechanics of colloidal suspensions formed in leachates, based on electrical double layer interactions in homogeneous particle swarms, could possibly provide more insight to the understanding and the determination of soil volume shrinkage and energetics of soil cracking (Phillip et al., 1984). Nevertheless, the developed relationship between FSDP and percent strain contributes to the understanding of soil behavior in landfill situations and has direct practical applications.

Before proceeding further with the possible application of the concepts tested and results obtained during the research program, it is worthwhile to examine the importance of synaerisis shrinkage as a process for crack formation, within the clay liner.

6.6.1 Crack Formation in Landfill Liners

In chapter three, causes and mechanisms of crack formation expected in a MSW landfill clay liner were reviewed. Thermal dessication is known to cause cracking in compacted clay soils exposed to harsh environments. Although synaerisis shrinkage has been found to be a major cause of cracking in various situations (Plummer and Gostin, 1981), direct observations of synaerisis crack formation within MSW landfills are not reported in literature.

Although, it is suspected that cracks form within landfills and directly contribute to the bulk permeability increases, collection of data to support this hypothesis from field observations is extremely difficult. Increased permeabilities across compacted clay liners point to the existence of discontinuties, which are capable of transporting leachate in large quantities.

Cracks within a clay liner would form under the following conditions:

- dessication cracks formed as a result of liquid loss if the liner is exposed to the environment; the majority of these cracks form before the actual operation of the landfill,
- cracks formed as a result of differential settlement of the compacted soil caused by synaerisis shrinkage (

here synaerisis shrinkage refers to the settlements produced by increased effective stresses resulting from clay-leachate interaction), induced by uneven generation of leachate within a landfill,

- Large pore discontinuities caused by volumetric shrinkage and separation of leachate-contacted soil from the main soil mass.

In addition, the possibility exists for clay liner damage (caused by differential settlements) and crack formation, during the initial stages of landfill operation, as a result of heavy equipment usage for solid waste compaction purposes.

The edges or the side slopes of a landfill would exhibit cracks formed as a result of the third mechanism mentioned above. In containment structures, with a bottom clay liner and embankments made of clay soil to prevent leachate migration, cracks would form in the clay-lined embankment, as a result of synaerisis shrinkage.

The synaerisis consolidation results gave ample evidence of what should be expected, if MSW leachates are allowed to get in contact with bentonite-slurry cut-off walls sometimes used as a leachate control measure. Despite the high and increasing rate of use of soil-bentonite (SB), backfill method of slurry trench construction, very little information is available concerning the performance of field installations (D'Appolonia, 1980). Although hazardous waste and municipal landfill designers have recommended their use, very few investigations have been conducted to evaluate their suitability for landfill situations. Given the high potential for synaerisis shrinkage of bentonite (sodium montmorillonite), SB cut-off walls would pose only restraint to migrating leachates. In landfills, SB cut-off walls should only be installed for the sole purpose of diverting groundwater away from the landfill. Direct contact with landfill leachates should always be avoided. However, if calcium montmorillonites are available, SB cut-off walls could become a viable alternative for any leachate control application.

6.6.2 Leachate Movement Across a Liner Containing Macropores

The relatively large cracks created either by dessication or synaerisis shrinkage of a compacted clay liner would occur in a fine-porous soil matrix. This matrix has a very low hydraulic conductivity and significant fluxes of leachate through the entire compacted soil mass are therefore only possible when continuous, large pores are present. Flow of liquids in clay soils containing cracks, fissures and slickensides is quite different from flow in more sandy soils, in which most soil pores contribute to liquid movement (Bouma, 1984).

Bouma et al. (1977), noted that liquid infiltration into soils containing cracks~~,~~ primarily occurs through these macropores, with infiltration through the soil matrix being relatively small. Quite a few approaches are available to model the flow through macropores or channels (Beven and Germann, 1982), of these the most popular is the two domain (channels/matrix) concept. A comprehensive review, to select a conceptual model capable of predicting leachate movement across the cracked clay liner, is beyond the scope of this study. However, an attempt was made to identify a simple analytical model that can be readily applied to the situation at hand. This would allow design engineers to visualize the seriousness of the problem.

Witherspoon et al. (1979), while studying hydraulic properties of rock fractures, noted that the hydraulic conductivity of a fracture could be determined by the following expression:

$$k_f = \frac{(2b)^3 \rho g}{12 \mu} \qquad 6.14$$

Where, (2b) was the aperture size of the fracture. This equation has certain limitations, in that, the fracture was assumed smooth, and opening width constant. For a crack in a clay soil with a variable opening width and a shrinking/swelling soil mass, a better approximation would be the modified Poiseuille equation proposed by Neuzil and Tracy (1981):

$$Q = L J \frac{\gamma}{12 \mu} \int b^3 (f^b) \, db \qquad 6.15$$

in which,

f(b) = the normalized aperture frequency distribution,

b = segment aperture,

L = the total length of the opening normal to the flow,

J = hydraulic gradient causing the flow Q.

The selection of appropriate frequency distribution, f(b), is empirical and should be based on synaerisis or dessication crack observations. When the crack is subjected to different kinds of stresses, caused by swelling or shrinking during leachate infiltration, the fracture aperture distribution changes. The closing and widening (or compression and extension) of the fractures as a result of swelling or shrinking could be accounted for by modifying the $\int b^3 f(b)\,db$ term in the above equation (Neuzil and Tracy, 1981).

Difficulties in obtaining the required data would limit the application of Equation 6.15 in sanitary landfill situations. However, the following set of simple equations, based on the channel/matrix concept proposed by Gray (1984), is worth checking for practical applicability.

For an open crack system, with clear smooth walls, equivalent permeability is given by;

$$K_e = \frac{2a}{b} \left\{\frac{a^2 g}{3\gamma}\right\} + k \qquad 6.16$$

Where,

a = half width of the crack, L,

b = crack spacing, L,

γ = kinematic viscosity of leachate, L^2/T,

k = hydraulic conductivity of an intact liner (or laboratory determined hydraulic conductivity of the soil), L/T.

This equation assumes the crack opening to be constant during leachate permeation and neglects the effect of either swelling or shrinking of the soil matrix. However, this could provide an estimate of the "worst situation" hydraulic conductivity, expected in a cracked clay liner. Gray (1984) estimated that cracks of 0.1 mm width, spaced at 1 m intervals, would increase the permeability from 10^{-10} m/s to 8.3×10^{-6} m/s (an increase of more than 3 orders of magnitude). The crack pattern suggested here more or less fits the dessication cracking and the Equation 6.12 should be able to approximate the initial hydraulic conductivity in a dessicated clay liner.

The situation with synaerisis cracks would be different. Large cracks may form at the edge of a leachate pool, which would drastically increase the bulk permeability. On the other hand, tiny subaerial cracks (Plummer and Gostin, 1981) would increase the hydraulic conductivity, k, in a manner similar to the magnitude suggested by Equation 6.16.

A more conservative approach is to assume that the cracks are infilled with a soil of known permeability.

Therefore, equivalent hydraulic conductivity in a clay mass, containing cracks caused either by dessication or synaerisis, could be obtained by mass balance.

Total flux = flux through soil matrix + flux through channels

$$k_e \; J \; A = k_c \; J \; A_c + k \; J \; (A - A_c)$$

The simplified equation is:

$$k_e = \frac{k_c \; A_c + k(A - A_c)}{A} \qquad 6.17$$

where,

k_c = hydraulic conductivity of the infill soil (if fine sand, 1×10^{-4} m/s),

A_c = x-sectional area normal to the flux,

A = total x-sectional area.

For the situation considered earlier (0.1 mm width cracks at 1 m spacing), Equation 6.17 yeilds a 1×10^{-7} m/s equivalent hydraulic conductivity value.

Equation 6.16 is normally used in geotechnical practice to estimate the hydraulic conductivity of a joint-intact block system (Lee et al. 1983). The application of either Eq. 6.16 or 6.17 for flow through a cracked compacted clay liner has not been reported in literature. Limited scope of this research prevented the attempt to verify experimentally any of the analytical equations mentioned here. However, considering the possible application of such analytical models for design purposes, further investigation of these models are worth considering. Moreover, field and experimental evidence of crack formation, pattern and extent of cracking within a clay liner are also needed, to substantiate the necessity for such an investigation.

SUMMARY AND CONCLUSIONS

1. MSW leachates, containing variable amounts of mono- and di-valent cations, are capable of causing shrinkage in high swelling clay soils. The magnitude of shrinkage depends upon the repulsive stresses present within the clay soil, and the characteristics of the leachate interacting with the soil.

2. Swelling pressure or the interparticle repulsive stress in a clay mineral mixture, at a given time, is highly dependent upon the percentage of each clay mineral present, the exchangeable and soluble cations in each clay mineral, the amount and type of cations present in the pore water. Clay soil mixtures high in Na montmorillonites exhibited the largest repulsive stress at a given void ratio. The Ca^{+2} version of montmorillonite did not swell very much in deionized water and was not capable of producing high repulsive stresses. Presence of high cation concentrations and di-valent cations within the pores prevent the development of high repulsive stresses. Ca montmorillonites (unlike kaolin), increase the concentration of Ca ions in the pore water. Therefore, in mixtures of similar proportions of Na montmorillonite/kaolin and Na montomorillonite/Ca montmorillonite, the Na/Ca montmorillonite mixture displayed the presence of a smaller repulsive stress.

3. The true effective stress, (σ^*), which controls the

volume change behavior of fully saturated soils, particularly the soils containing higher percentages of Na montmorillonite, is given by the equation:

$$\sigma^* = \sigma - u_w - (R-A)$$

MSW leachates primarily affect the (R-A) stress. When the soil is permeated with MSW leachate, cation concentration in the pore water increases. This results in the contraction of electric double layers surrounding the clay particle, and a subsequent decrease in the interparticle repulsive stress, R. The reduction in double-layer thickness allows clay particles to come close to each other, reducing the interparticle distance. Therefore, the attractive stress A increases. However, the increase in A is less significant than the reduction in R (magnitude of A is inversely proportional to the third power of interparticle half distance, d). End result is a reduction in (R-A) stress and a net increase in effective stress, which eventually causes the synaerisis shrinkage of the soil.

4. The magnitude of synaerisis shrinkage depends upon the magnitude of σ^* change, and, therefore, on the net (R-A) reduction. Initial (R-A), before leachate addition, is controlled by the type of clay mineral, exchangeable cations, and the pore water cation concentration. Changes

incurred in the concentration and type of cations in pore water, by leachate, controls the post-leachate (R-A) stress. Divalent cation, Ca^{+2}, even at moderate concentrations (0.045M), is capable of reducing double layer thicknesses and causing a 80-90% change in the net (R-A) stress. Replacement of Na^{+} ions in the exchange complex, and in bound water by cation Ca^{+2}, is the primary cause of double layer diminution, in Na montmorillonite soil systems.

5. In a di- and mono-valent cation mixture, as in the case of MSW leachate, divalent cation has the predominant influence on double layer thickness and repulsive stress. This was evident in both synaerisis consolidation and sedimentation experiments, as both percent strain and FSDP are found to have a direct relationships with the concentration of calcium in leachate. Higher calcium concentrations in leachate caused higher percent strain for a given soil, at a particular void ratio, and higher FSDP values. Among the chemicals tested, in general, synaerisis shrinkage effect increases in the order, acetic acid, Na^{+}, Ca^{++}.

6. Magnitudes of soil-leachate parameters, percent strain and FSDP, are controlled by the effect of leachate upon the interparticle repulsive and attractive forces. The magnitude of sedimented volume is a function of the net interparticle repulsion. Leachates reduce the net interparticle (R-A)

stress and produce a smaller sedimented volume (larger FSDP) and also cause synaerisis shrinkage in a soil sample prepared by artificial sedimentation. Furthermore, a unique empirical relationship exist between the two parameters, FSDP and percent strain, proving that synaerisis shrinkage and sedimentation in a leachate environment are both controlled by the same mechanism. An estimate of this relationship is:

$$\log(\text{percent strain}) = -\,0.6582 + 0.052\,(\text{FSDP})$$

7. The sedimentation test, although simple, is highly reproducible and provides valid results within a reasonable time frame. Consolidation experiments take 6 to 8 weeks to perform. The sedimentation test provides results within 24 hours. By determining the FSDP value for a given clay soil-leachate system, the above empirical relationship could be used to predict the synaerisis shrinkage expected in a long-term, consolidation experiment. The sedimentation test, therefore, could be considered an "index" test to identify the compatibility of clay soils with leachate.

8. FSDP, or the sedimentation process, reflects only the volume shrinkages caused by changes in interparticle physico-chemical forces. FSDP is not capable of predicting the shrinkages expected from other than causes related to

changes in interparticle forces (an example not predicted, is the dissolution of clay minerals and subsequent collapse of soil fabric, caused by high acetic acid concentrations).

9. The double layer theories and the effective stress concept, as adapted here, are capable of adequately describing the observed synaeresis shrinkage and sedimentation behaviour of pure clay mineral mixtures comprised of Na montmorillonite, kaolinite, Ca montmorillonite and illite.

10. All three natural soils contained less than 25% montmorillonites in the clay fraction. In addition, the predominant exchangeable cation was Ca^{+2} (more than 65% in all soils), exchangeable Na^{+} being less than 18% for all three soils. These soils can be categorized as non-swelling soils. FSDP values for these soils are insignificant. Subsequently, very little shrinkage was observed during synaerisis consolidation experiments. As predicted by FSDP determinations and theoretical considerations, bentonite-modified (10% bentonite by weight) soils exhibited volume shrinkage during leachate charging.

11. The "standard leachate", as employed in this work, comprising of Ca^{+2} = 0.045M, Na^{+} = 0.017M and HAc = 250 mg/L, is capable of producing volume shrinkages which are comparable with the shrinkages caused by highly concentrated

leachates. The "standard leachate" was capable of causing 80% to 95% of the observed synaerisis shrinkage, with 0.45M Ca^{+2} solutions and 30% NaCl brine. Therefore, the proposed "standard leachate" could be used to perform MSW leachate-soil compatibility studies and direct leachate permeability tests.

12. Mixing Ca^{+2} ions (as $CaCl_2$) with the pore water, tends to stabilize the volume shrinkage behavior of tested soils. Only sufficient amounts of Ca^{+2} ions to neutralize the net repulsive stress in the soil need to be added, to prevent the volumetric shrinkages attributed directly to synaerisis. With higher Na montmorillonite percentages in a soil, higher concentration of Ca^{+2} should be added. The natural soils tested do not need Ca^{+2} stabilization.

13. The volume shrinkage in a consolidation cell is caused by "synaerisis" (change in internal forces within the clay soil-electrolyte system), rather than osmotic consolidation (the consolidation cell acting as an osmotic cell). Once liquid in the leachate reservoir is changed from deionized to standard leachate, a hydraulic gradient across the sample has to be provided, before any volume change was recorded. Samples charged with the same NaCl solution (25,000 mg/L) showed different behaviors, depending upon the hydraulic head. Of the two soil samples, only the sample with the hydraulic head forcing mass flux of leachate across the soil

showed significant volume shrinkage. The soil samples do not behave as osmometers nor do they lose water due to osmosis.

14. Since the leachate manages to remove the bound water in the double layer, the post-leachate soil consolidates faster (reduction in t_{90}) and to a smaller void ratio for a given load increment. Therefore, the compression index of leachate treated soils (based on the post-leachate load increment strain behavior) are higher than the non-treated soils. Increasing Ca^{+2} concentration in leachate produces faster consolidation rates. Therefore, the consolidation coefficient, C_v, increases with increasing Ca^{+2} concentrations. Large changes in particle arrangements, either EF or FF, are not observed (in the electron microscopic studies) as a result of leachate interactions.

15. The percent strain resulting from synaerisis shrinkage is not strongly affected by surcharge pressures in the pressure range, 40 to 160 kPa. However, as the void ratio is also dependent upon the interparticle stresses, among other factors, different soils, compacted at a given surcharge pressure, showed different final void ratios. Swelling soils high in Na montmorillonite, showed higher void ratios, and higher shrinkages. However, at high surcharge pressures (560 kPa) and low void ratios, large σ^* changes are necessary to cause volume shrinkages. Therefore, at very high surcharge pressures leachates cause smaller synaerisis shrinkages.

16. Hydraulic conductivity (k), calculated from consolidation results, obtained before and after leachate permeation, did not show a significant change. This has also been confirmed by direct permeability tests. Hydraulic conductivity, being controlled by pore size distribution, is sensitive to changes in pore sizes. The removal of bound water, due to synaerisis, causes an increase in the size of pore spaces, but the subsequent consolidation and closing of pores would nullify this effect. If the soil was not allowed to shrink or consolidate due to leachate addition (incidently, in the field, consolidation is bound to happen), then an increase in pore sizes could occur. The removal of water, due to consolidation, could cause an increase in the leachate flowing out during leachate charging. The increase in hydraulic conductvity, observed after the permeant in the leachate reservoir was changed from deionized to leachate, was partly due to this action, and should not be confused with actual permeation. At least 3 to 6 weeks (time dependent on the type of soil and permeant characteristics) is necessary to overcome this mechanism of liquid flow. The permeability tests should be performed only after the volume shrinkage has been completed, or else liquid flow caused by synaerisis consolidation should be calculated and subtracted from the total flow.

During brine permeation, at low hydraulic gradients, rapid volume shrinkages and sudden increases in liquid flow was observed. Once volume shrinkage ceased, liquid flow reduced to a minimum, indicating that the initial sudden increase in flow was caused by none other than the squeezing out of liquid during the rapid synaerisis consolidation of the soil.

It was necessary to keep a high enough (in this case 80 kPa) vertical stress to prevent lateral shrinking of soil. If the soil moves away from the consolidation cell wall, MSW leachate is capable of causing the "breakthrough phenomenon" reported in literature. Therefore, in performing a valid permeability test, volume change should also be monitored (particularly in the case of high swelling soils), together with the the measurement of volumetric liquid flow.

RECOMMENDATIONS

This research identified certain limitations in the current practice of MSW landfill liner design and ways of alleviating these problems. Furthermore, several areas at which further research efforts should be directed, are also identified.

The following steps are proposed to be included in a rational design procedure for MSW landfill liners:

- Hydraulic conductivity (k) of the soil material,
    - if available, should be considered only as a first approximation of the hydraulic conductivity across the clay liner,
    - if not available, expensive direct permeability testing should not be considered in an attempt to evaluate k. The k value, calculated from consolidation results obtained at the field bulk density, is sufficient for a first approximation.
- Volume Change Behavior
    - sedimentation tests should be performed in order to understand the compatibility of the soil material with leachate,
    - if volume changes are expected, indicated by FSDP value, synaerisis consolidation tests need to be performed to determine the magnitude of volume shrinkage.
- Design Modifications
    - if synaerisis shrinkages are observed during

synaerisis consolidation experiments, the soil material may need to be treated with Ca.

This investigation attempted to shed light upon the importance of considering volume shrinkage and subsequent crack formation, as a means of explaining increased permeability across a compacted clay liner. However, in order to completely understand the contribution of volume shrinkage and crack formation, to bulk permeability, specifically the synaerisis crack formation, further research is needed in the following areas:

- laboratory studies to relate synaerisis shrinkage to increased permeability with field verification,
- conceptual models of crack formation due to synaerisis and flow through such cracks,
- flow through deformable (shrinking/swelling) cracks,
- collection and analysis of data, on leakage from lined landfill sites,in order to understand the mechanisms causing increased permeability across landfill liners,
- further research into the flow of leachates under low hydraulic gradients. Brine permeation at low gradients, and implication in brine pond design need to be reviewed in light of the observations made during this investigation.

BIBLIOGRAPHY

Ali, P. (1965), "The Mechanisms of Swelling in Clay Soils", MSc Thesis, Department of Civil Engineering, The University of Alberta, Edmonton, Alta.

Allison, L.E.(1947), "Effect of Microorganisms on Permeability of Soil Under Prolonged Submergence", Soil Science, vol.63, pp. 439-450.

American Public Health Association, American Water Works Association and the Water Pollution Control Federation (1976), "Standard Methods for the Examination of Water and Wastewater", Fourteenth Edition.

Anderson, D.(1982), "Does Landfill Leachate Make Clay Liners More Permeable", Civil Engineering, ASCE, Sept. 1982, pp. 66-69.

Anderson, D., and Brown, K.W.(1981), "Organic Leachate Effects on the Permeability of Clay Liners", Land Disposal of Hazardous Wastes, Proceedings of the Seventh Annual Research Symposium, U.S., EPA 600/9-81-002b, Washington, D.C., pp. 119-130.

Anderson, D., Brown, K.W., and Green, J. (1982), "Effect of Organic Fluids on the Permeability of Clay Soil Liners", Land Disposal of Hazardous Wastes, Proceedings of the Eigth Annual Research Symposium, U.S., EPA 600/9-82-002, Washington, D.C., pp. 179-190.

Anderson, J.R. and Dornbush, J.N.(1967), "Influence of Sanitary Landfills on Groundwater Quality", Journal, American Water Works Association, Vol.59, pp. 457-470.

Anderson, W.C., and Glysson, E.A.(1982), "Land Disposal Technology Liners" Hazardous Wsate Management - Course Notes, ASCE Continuing Education Course, July 1982.

Apgar, M.A. and Langmuir, D.(1971), "Groundwater Pollution Potential of a Landfill Above the Water Table", Groundwater, Vol.9, pp. 6-11.

Arnold, P.W.(1978), "Surface-Electrolyte Interactions" IN The Chemistry of Soil Constituents, Eds. D.J. Greenland and M.H.B. Hayes, Wiley-Interscience Publication, pp. 304-404.

Arulanandan, K., Loganathan, P. and Krone, R.B.(1975), "Pore and Eroding Fluid Influences on the Surface Erosion of a Soil", Journal of the Geotechnical Engineering Division, ASCE, pp. 53-66.

Aylmore, L.A.G., and Quirk, J.P.(1959), "Swelling of Clay-Water Systems" Nature, (London) Vol.183, pp. 1752-1753.

Aylmore, L.A.G., and Quirk, J.P.(1962), "The Structural Status of Clay Systems" Clays and Clay Minerals, Proc. Ninth International Conference, pp. 104-130.

Bache, B.W.(1977), "The Measurement of Cation Exchange Capacity of Soils", Potash Review, #1/1977, pp. 1-9.

Baedecker, M.J. and Back, W.(1980), "Hydrogeological Processes and Chemical Reactions at a Landfill", Groundwater, Vol.17, #5, pp. 429-437.

Bailey, W.A.(1965), "The Effects of Salt on the Consolidation Behavior of Saturated Remoulded Clays", Phase Report #3, USAE Waterways Experiment Station, Vicksburg, Miss., Report #3-101, pp. 161.

Baker, B. L., Parker. and Peake, E. (1981), Chemical Composition of CLOVER BAR landfill leachates, Kananaskis Center for Environmental Research, University of Calgary, Calgary, Alberta.

Balasubramonian, B.I.(1972), "Swelling of Compaction Shale" Ph D Dissertation, Department of Civil Engineering, University of Alberta, Edmonton, Alberta, Spring 1972.

Barshad, Isaac. (1952), "Factors Affecting the Interlayer Expansion of Vermiculite and Montmorillonite with Organic Sub~~s~~tances", Soil Science Society of America Proceedings, Vol.17, pp. 176-182.

Baver, L.D., and Winterkorn, H.F.(1935), "Sorption of Liquids by Soil Colloids", Soil Science , Vol.40, pp. 403-419.

Bayrock, L.A., and Hughes, G.M.(1962), "Surficial Geology of the Edmonton District, Alberta", Research Council of Alberta, Preliminary Report, #62-6.

Bear, J.(1972), "Dynamics of Fluids in Porous Media", American Elesevier, New York.

Bear, J.(1979), Hydraulics of Groundwater, McGraw-Hill, New York, 567 p.

Been, K. and Sills, G.C.(1981), "Self Weight Consolidation of Soft Soils", Geotechnique, Vol.31, #4, p. 519.

Been, K. and Sills, G.C.(1984),"Escape of Pore Fluid from

Consolidating Sediment" in Transfer Processes in Cohesive Sediment System, Eds., W.R. Parker and D.J.J. Kinsman, Plenum Press, New York, pp. 109-125.

Berndt, R.D., and Coughlan, K.J.(1976),"The nature of Changes in Bulk Density with Water Content in a Cracking Clay" Australian Journal of Soil Research, Vol.15, pp. 27-37.

Beven, K. and Germann, P.(1982), "Micropores and Water Flow in Soils", Water Resources Research, Vol.18, pp. 1311-1325.

Black, Charles Allen, ed. (1965), Methods of Soil Analysis, Vol.1&2 (Agronomy 9), American Society of Agronomy, Madison, Wis.

Blackmore, A.V. and Miller, R.D.(1961), "Tactoid Size and Osmotic Swelling in Calcium Montmorillonite", Soil Science Society of America Proceedings, Vol.25, pp. 169-173.

Bohn, H.L., McNeal, B.L. and O'Connor, G.A.(1979), Soil Chemistry, A Wiley-Interscience Publication, New York, 329 p.

Bohor, B.F. and Hughes, R.E.(1971), "Scanning Electron Microscopy of Clays and Clay Minerals", Clays and Clay Minerals, Vol.19, pp. 49-54.

Bolt, G.H.(1955), "Analysis of the Validity of the Gouy-Chapman Theory of the Electric Double Layer", Journal of Colloid Science, Vol.10, pp. 206-218.

Bolt, G.H. and Miller, R.D.(1955), "Compression Studies of Illite Suspensions", Soil Science Society of America Proceedings, Vol.20, pp. 285-288.

Bolt, G.H.(1956), "Physical-Chemical Analysis of the Compressibility of Pure Clay", Geotechnique, vol.6, #2, pp. 86-93.

Bolt, G.H. and Bruggenwert, J.(1976), "Soil Chemistry - Basic Elements", Developments in Soil Sciences 5A, Elsevier, Amsterdam, 281 p.

Bonneau, M. and Souchier, B.(1982), Constituents and Properties of Soils, Academic Press, New York, 496 p.

Bouma, J.(1984), "Using Soil Morphology to Develop Measurement Methods and Simulation Techniques for Water Movement in Heavy Clay Soils", Proceedings, ISSS Symposium on Water and Solute Movement in Heavy Clay Soils, Eds., J. Bouma and P.A.C. Raats, ILRI Publication

37, Wageningen, The Netherlands, pp. 298-315.

Bouma, J., Jongerius, A., Boersma, O., Jager, A. and Schoonderbeek, D.(1977), "The Function of Different Types of Macropores During Saturated Flow Through Four Swelling Soil Horizons", Soil Science Society of America Journal, Vol.41, pp. 945-950.

Brown, K.W. and Anderson, D.(1980), "Effect of Organic Chemicals on Clay-Liner Permeability" Proceedings of the Sixth Annual Research Symposium, U.S., EPA - 600/9-80-010, Washington, D.C., pp. 123-134.

Burst, J.F.(1965), "Subaqueously Formed Shrinkage Cracks in Clay", Journal of Sedimentary Petrology, Vol.35, pp. 348-353.

Chamberlain, E.J.(1981), "Frost Susceptibility of Soil,-Review of Index Tests," US Army Cold Regions Research and Engineering Laboratory (CRREL), Monograph 81-2, Hanover, New Hampshire.

Chapman, H.D.(1965), "Cation Exchange Capacity", in Methods of Soil Analysis, Agronomy #9, Part 2, C.A. Black, ed., pp. 891-901.

Chattopadhyay, P.K. (1972), " Residual Shear Strength of Some Pure Clays", PhD dissertation, Department of Civil Engineering, University of Alberta, Fall 1972.

Chian, E.S.K.(1976), "Stability of Organic Matter in Landfill Leachates" Water Research vol.11, pp. 225-232.

Chian, E.S.K. and Dewalle, F.B.(1976), "Characteristics of Landfill Leachates and Their Treatment", Journal of the Environmental Engineering Division, ASCE, Vol.102, EE2.

Clark, T.P.(1975), "Survey of Groundwater Protection Methods for Illinois Landfills", Groundwater, vol.13, # 4.

Collins, K. and McGowan(1974), "The Form and Function of Microfabric Features in a Variety of Natural Soils", Geotechnique, Vol.24, #2, pp. 223-254.

Collis-George, N. and Bozeman, J.M.(1970), "A Double Layer Theory for Mixed Ion Systems as Applied to the Moisture Content of Clays Under Restraint", Australian Journal of Soil Research, Vol.8, pp. 239-258.

Crim, R.G., Shepherd, T.A., and Nelson, J.D. (1979), "Stability of Natural Clay Liners in a low pH Environment", Proceedings of the Second Symposium on Uranium Mill Tailings Management, Colorado State University, pp. 41-54.

Czeratzki, W., and Frese, H.(1958), "Importance of Water in Formation of Soil Structure", Water and its Conductance in Soils, Special Report #40, Highway Research Board.

D'Appolonia, D.J.(1980), "Soil-Bentonite Slurry Trench Cutof thefs", Journal of the Geotechnical Engineering Division, ASCE, Vol.106, #GT4, pp. 399-417.

Daniel, D.E.(1984), "Selected Case Histories of Field Performance of Compacted Clay Liners", Journal of the Geotechnical Engineering Division, ASCE, Vol.110, #2, pp. 285-300.

Davies, O.L. ed.(1979), The Design and Analysis of Industrial Experiments, Second Edition, Longman Inc., New York, 636 p.

Davies, J.S., and Coleman, R.C.(1980), "Microbiological Processes Within a Landfill", A Paper Presented at the Sanitary Landfill Workshop held in Alberta Environment Center, Vegreville, Alta.

Dawson, R.F.(1965), Discussion on Paper by W.G.Holtz and H.J.Gibbs in ASCE Transactions, vol.121, pp. 641-663.

De Jong, E., and Warkentin, B.P.(1965), "Shrinkage of Soil Samples with Varying Clay Concentrations", Canadian Geotechnical Journal, vol.2, #1, pp. 16-22.

Delage, P. and Lefebvre, G.(1983), Discussion on "Double Layer Theory and Compressibility of Clays", by A. Sridharan and M.S. Jayadeva(1982), Geotechnique, Vol.33, p. 461.

Draper, N.S. and Smith, H.(1981), Applied Regression Analysis, Second Edition, John Wiley and Sons, New York, 709 p.

Dunn, R.J.(1983), "Hydraulic Conductivity of Soils in Relation to the Subsurface Movement of Hazardous Wastes", PhD Dissertation, Department of Civil Engineering, University of California, Berkeley, Spring 1983.

Dunn, R.J., and Mitchell, J.K.(1984), "Fluid Conductivity Testing of Fine Grained Soils", Journal of the Geotechnical Engineering Division, ASCE, Vol.110, #11, pp. 1648-1665.

Elliot, G.L.(1977), "Amelioration of Tunnelling Susceptible Soils in the Hunter Valley, New South Wales, by Modification of Clay Double-Layer Interactions", IN Proceedings of the Symposium on Dispersive Clays, Related Piping and Erosion in Geotechnical Projects, ASTM #623, J.L. Sherard and R.S. Decker, Eds., Chicago, Ill., pp. 110-120.

Emerson, W.W.(1963), "The Swelling of Na-montmorillonite due to Water Adsorption" Australia Journal of Soil Research, Vol.1, pp. 129-143.

Ezozy, Y.(1980), "Adsorption of Cd and Co on Montmorillonite as a function of Solution composition", Clays and Clay Minerals, vol.28, pp. 311-318.

Falconer, J.G., and Mattson, S.(1933), "The Laws of Colloidal Behaviour; 13- Osmotic Imbibition", Soil Science, vol.36, pp. 317-328.

Farrar, D.M. and Coleman, J.D.(1967), "The Correlation of Surface Area with Other Properties of Nineteen British Clay Soils", Journal of Soil Science, Vol.18, #1, pp. 118-124.

Fink, D.H. and Nakayama, F.S.(1972), "Equation for Describing the Free-Swelling of Montmorillonite in Water", Soil Science, vol.114, #5, pp. 355-358.

Fireman, M.(1944), "Permeability Measurements on Disturbed Soil Samples", Soil Science, Vol.58, p. 337.

Folkes, D.J.(1983), "Control of Contaminant Migration by the Use of Liners", Canadian Geotechnical Journal, Vol.19, pp. 320-329

Fox, D.J. and Guire, K.E.(1976), Documentation for MIDAS, Third Edition, Statistical Research Laboratory, University of Michigan, 203 p.

Freeze, R.A. and Cherry, J.A.(1979), " Groundwater", Prentice-Hall Inc., New Jersey.

Fuller, W.H. and Warrick, A.W.(1985), "Soils in Waste Treatment and Utilization", Vol. II, CRC Press, Florida, 235 p.

Fung, R.(1980), Protective Barriers for Containment of Toxic Materials, Pollution Technology Review, #66, Noyes Data Corporation, New Jersey.

Fungaroli, A.A. and Steiner, R.L.(1971), "Laboratory Study of the Behavior of a Sanitary Landfill", Journal, Water Pollution Control Federation, Vol.43, #2, pp. 256-267.

Garland, G.A. and Mosher, D.C.(1975), "Leachate Effects from Improper Land Disposal", Waste Age, Vol.6, pp. 42-48.

Gast, R.G.(1977), "Surface and Colloid Chemistry" in *Minerals in the Soil Environment*, Eds. J.B. Dixon and S.W. Weed, pp. 27-73.

Geswein, A.J.(1975), *Liners for Land Disposal Sites: An Assesment*, EPA/530/SW-137, U.S. Environmental Protection Agency, Washington, DC, 66pp.

Gee, G.W., Campbell, A.C., Opitz, B.E., and Sherwood, D.R.(1980), "Interactions of Uranium Mill Tailings with Morton Ranch Clay Liner and Soil Material", *Proceedings of the Third Uranium Mill Tailings Management Symposium*, Colorado State University, pp. 333-354.

Genetelli, E.J., Cirello, J. (1976), Eds., *Gas and Leachate from Landfills - Formation, Collection and Treatment*, EPA 600/9-76-004.

Gillott, J.E.(1969), "Study of Fabric of Fine-Grained Sediments with the Scanning Electron Microscope", *Journal of Sedimentary Petrology*, Vol.39, pp. 90-105.

Gillott, J.E.(1980), "Use of Scanning Electron Microscope and Fourier Methods in Characterization of Microfabric and Texture of Sediments", *Journal of Microscopy*, Vol.120, pp. 261-270.

Glasstone, S.(1978), *Textbook of Physical Chemisty*, Second Edition, D. van Nostrand Publications, New York, 921 p.

Gorden, B.B., and Forrest, M.(1981), "Permeability of Soils Using Contaminated Permeant", IN *Permeability and Groundwater Transport*, ASTM STP 746, T.F.Zimmie and C.O.Riggs, Eds., American Society for Testing and Materials, pp.101-120.

Gravenor, S.E. and Bayrock, L.A.(1962), "Glacial Deposits of Alberta", Report #G45, Department of Geology, University of Alberta.

Gray, D.H.(1984), "The Properties and Behavior of Compacted Clay and its Interaction with Landfill Leachate", Course Notes, *Short Course on Geotechnical Engineering of Land Disposal Systems*, University of Wisconsin - Extension, October 15-19, 1984

Green, W.J., Lee. G.F., and Jones, R.A.(1979), *Impact of Organic Solvents on the Integrity of Clay Liners for Industrial Waste Disposal Pits; Implications for Groundwater Contamination*, EPA Report PB 81-213423.

Green, W.J., Lee, G.F. and Ann Jones, R. (1981), "Clay Soils Permeability & Hazardous Waste Storage"*Journal Water Pollution Control Federation* , Vol.53, # 8, pp.

1347-1354.

Green, W.J., Lee, G.F., Jones, R.A. and Palit, T.(1983), "Interaction of Clay Soils with Water and Organic Solvents: Implications for the Disposal of Hazardous Wastes", Environmental Science & Technology, Vol.17, pp. 278-282.

Griffin, R.A. and Shimp, N.F.(1975), "Interaction of Clay Minerals and Pollutants in Municipal Leachate", Water Reuse, Vol.2, pp. 801-811.

Griffin, R.A., Cartwright, K. Shimp, N.F., Steel, J.D., Ruch, R.R., White, W.A., Hughes, G.M. and Gilkeson, R.H.(1976), "Attenuation of Pollutants in Municipal Landfill Leachate by Clay Minerlas: Part I. Column Leaching and Field Verifications", Illinois State Geological Survey, Environmental Geology Notes #78 , 34 p.

Grim, R.E.(1968), Clay Mineralogy, Second Edition, McGraw Hill, New York.

Haji-Djafari, S., Anommaria, P.E. and Crouse, H.L.(1981), "Attenuation of Radionuclides and Toxic Elements by In-Situ Soils at a Uranium Tailings Pond in Central Wyoming", IN Permeability and Groundwater Transport, ASTM STP #746, T.F. Simmie and C.O. Riggs, Eds., American Society for Testing and Materials, Philadelphia.

Haji-Djafari, S. and Wright, J.C.(1983), "Determining th Long Term Effects of Interactions Between Waste Permeants and Porous Media", Hazardous and Industrial Solid Waste Testing: Second Symposium, ASTM STP 805, R.A. Conway and W.P. Gulledge, Eds., American Society for Testing and Materials, pp. 246-264.

Hamilton, A.B.(1966), "Freezing Shrinkage in Compacted Clays", Canadian Geotechnical Journal, Vol.3, #1, pp. 1-17.

Hamilton, J.J.(1980), "Behaviour of Expansive Soils in Western Canada", Proceedings of the Fourth International Conference on Expansive Soils, vol.2, Denver, Colorado, pp. 815-833.

Hardcastle, J.H. and Mitchell, J.K.(1974), "Electrolyte Concentration-Permeability Relationships in Sodium Illite-Silt Mixtures", Clays and Clay Minerals, Vol.22, pp. 143-154.

Harrop-Williams, K.O.(1982), "Clay Liner Permeability A Prbabilistic Description" in Hazardous Waste Management for the 80's, Eds., R.M. Sykes and O.J. Sproul, pp. 307-313.

Haug, M.D.(1985), "Design and Construction of a Pre-Hydrated Sand-Bentonite Liner to Contain Brine", Paper Presented at the Canadian Society for Civil Engineering, Annual Conference, Saskatoon, SK.

Haxo, H.E. (1981), "Durability of Liner Materials for Hazardous Waste Disposal Facilities" Land Disposal of Hazardous Wastes, Proceedings of The 7th Annual Research Symposium, EPA 600/9-81-0026 US EPA, Cincinnati, Ohio, pp. 140-156

Haxo, H.E., White, R.M., Haxo, Paul D. and Fong, M.A. (1983), "Liner Materials Exposed To Municipal Solid Waste Leachate" Project Summary, EPA-600/S2-82-097, US EPA, Cincinnati, Ohio.

Heller-Kallai, L.(1981), "Clay-Salt Interactions" Proceedings, Seventh International Clay Conference, Held in Bologna & Paris, Eds; H. van Olphen and F. Veniale, p.127.

Hillel, D.(1971), Soil and Water - Physical Principles and Processes, Academic Press, New York.

Holtz, W.G., and Gibbs, H.J.(1956), "Engineering Properties of Expansive Clays", Transactions, American Society of Civil Engineering, vol.121, pp. 641-663.

Hurst, R.E.(1970), "Chemicals Find Growing Use in Oil Fields", Chemical and Engineering News, Vol.48, March, p. 10.

Imai, G.(1980), "Settling Behavior of Clay Suspensions", Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, Vol.20, #2, pp. 61-77.

Imai, G.(1981), "Experimental Studies on Sedimentation Mechanism and Sediment Formation of Clay Materials", Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, Vol.21, #2, pp. 7-20.

Johansen, O.J., and Carlson, D.A.(1976), "Characterization of Sanitary landfill Leachates", Water Research, vol.10, pp. 1129-1134.

Katzir, M. (1969), "Stabilization of active clay by injection and replacement methods" Proceedings, Second International Research & Engineering Conference on Expansive Soils, pp. 431-434.

Kearns, R.L., and Mankin, C.J.(1968), "Structural Charge Site Influence on the Interlayer Hydration of Expandable Three Sheet Minerals", Clays and Clay Minerals, vol.16, pp. 73-81.

Kemper, W.D. and Quirk, W.P.(1970), "Graphic Presentation of a Mathematical Solution for Interacting Double Layers", Soil Science Society of America, Proceedings, Vol.34, pp. 347-350.

Kingsbury, G.L., and Truesdale, R.S.(1985), "Clay-Chemical compatibility and Permeability Testing: A Review", Eleventh Annual Research Symposium on Land Disposal, Remedial Action, Incineration and Treatment of Hazardous Waste, Cincinnati, Ohio.

Kjeldsen, P. and Christensen, T.H.(1984), "Soil Attenuation of Acid Phase Landfill Leachate", Waste Management and Research, Vol.2, pp. 247-263.

Klute, A.(1965), "Laboratory Measurement of Hydraulic Conductivity of Saturated Soil", In "Methods of Soil Analysis", American Society of Agronomy, Monograph#9, pp. 210-221.

Klute, A.(1972), "The Determination of the Hydraulic Conductivity of Unsaturated Soils", Soil Science, vol.113, #4, pp. 264-276.

Komex Consultants Ltd.(1985), An Evaluation of the Effect of Brine on a Compacted Clay Till Liner, Report to Alberta Environment, 61 p.

Kruyt, H.R.(1952), Colloid Science, Vol.1, Elsevier Publications, Amsterdam.

Krynine, D.P. and Judd, W.R.(1957), Principles of Engineering Geology and Geotechnics, McGraw-Hill Civil Engineering Series, McGraw-Hill, New York.

Kynch, G.J.(1952), "A Theory of Sedimentation", Transactions, Faraday Society, Vol.48, pp. 166-176.

Ladd, C.C.(1960), "Mechanisms of Swelling of Compacted Clay", Bulletin, Highway Research Board, #245, pp. 10-26.

Lambe, T.W.(1951), Soil Testing for Engineers, John Wiley & Sons, New York.

Lambe, T.W.(1953), "The Structure of Inorganic Soil", Proceedings, American Society of Civil Engineers, Vol.79, seperate 315.

Lambe, T.W.(1954), The Permeability of Fine-Grained Soils, American Society for Testing and Materials, Special Technical Publication #163, pp. 56-67.

Lambe, T.W.(1960), "A Mechanistic Picture of the Shear Strength in Clays", Proceedings, Conference on Shear Strength of Cohesive Soils, ASCE, Boulder, pp. 555-580.

Lambe, T.W.(1958a), "The Structure of Compacted Clay", Journal of the Soil Mechanics and Foundations Division, ASCE, vol.84, SM2, paper #1654.

Lambe, T.W.(1958b), "The Engineering Behavior of Compacted Clay", Journal of the Soil Mechanics and Foundations Division, ASCE, Vol.84, SM2, paper #1655.

Lambe,T.W., and Whitman, R.V.(1969), Soil Mecahnics, John Wiley & Sons, Inc., New York.

Leskiw, G.(1981), "Elements of Engineering Design for a Sanitary Landfill", Waste Management Series, Publication #4, Alberta Environment, 53 p.

Lee, G.F. and Jones R.A.(1984), "Is Hazardous Waste Disposal in Clay Vaults Safe?", Journal, American Water Works Association, Vol.76, pp. 66-73.

Lee, I.K., White, W. and Ingles, O.G.(1983), Geotechnical Engineering, Pitman Publishers, Boston, 508 p.

Lindsay, J.D. and Scheelar, M.D.(1972), "Soil Surrey for Urban Development, Edmonton, Alberta", Research Council of Alberta, Report #72-7.

Loughry, F.G.(1973), "The Use of Soil Science in Sanitary Landfill Selection and Management", Geoderma, Vol.10, pp. 31-39.

Lu, J.C.S., Eichenberger, B. and Stearns, R.J.(1985), "Leachate from Municipal Landfills", Pollution Technology Review #119, Noyes Publications, 453 p.

Lutz, J.F., and Kemper, W.D.(1959), "Intrinsic Permeability of Clay as Affected by Clay-Water Interactions", Soil Science, Vol.88, pp. 83-90.

Macdowell, C.(1956), "Interrelationship of Load, Volume Change and Layer Thicknesses of Soils to the Behaviour of Engineering Structures", Proceedings, Highway Research Board, #245, pp. 10-26.

Macey, H.H.(1942), "Clay-Water Relationship and the Internal Mecahnism of Drying" Transactions, British Ceramic

Society, vol.41, pp.73-121.

Malcolm, R.L., Leenheer, J.a., and Weed, S.B.(1975), "Dissolution of Aquifer Clay Minerals During Deep Well Disposal" Proceedings, of the International Clay Conference, p.477.

Marsland, A., Prince, A. and Love, M.A.(1982), "The Role of Soil Fabric Studies in the Evaluation of the Engineering Parameters of Off-Shore Deposits", Proceedings, Third International Conference on Behavior of Offshore Structures, M.I.T. Boston, Mass.

Matrecon,Inc.(1980), Lining of Waste Impoundment and Disposal Facilities, EPA SW-870.

McKeague, J.A.(1978), Editor, "Manual on Soil Sampling and Methods of Analysis", Prepared by Subcommittee of Canada Soil Survey on Methods of Analysis, Soil Research Institute, Ottawa, 212 p.

McNeal, B.L. and Coleman, N.T.(1966), "Effect of Solution Composition on Soil Hydraulic Conductivity", Soil Science Society of America, Proceedings,Vol.30, pp. 308-316.

McRoberts, E.C. and Nixon, J.F.(1976), "A Theory of Soil Sedimentation", Canadian Geotechnical Journal, Vol.13, pp. 294-310.

Mehta, P.K., and Hu, F.(1978), "Further Evidence for Expansion of Ettrigite by Water Adsorption" Journal of the American Ceramic Society, Discussion and Notes, pp. 179-180.

Mesri, G. and Olson, R.E.(1971a), "Mechanisms Controlling the Permeability of Clays", Clays and Clay Minerals, vol.19, #3, pp. 151-158.

Mesri, G. and Olson, R.E.(1971b), "Consolidation Characteristics of Montmorillonite", Geotechnique, Vol.21, #4, pp. 341-352.

Metcalf, J.B.(1964), "Some Engineering Aspects of Soil Stabilization" In, Mechanics of Soil Stabilization, Proceedings of a Colloquium held at Syndal, Victoria, pp. D1-4 - D1-21.

Michaels, A.S., and Lin, C.S.(1954), "The Permeability of Kaolinite to Various Fluids", Industrial and Engineering Chemistry, vol.46, pp.1239-1246.

Michaels, A.S. and Bolger, J.C.(1962), "Settling Rates and Sediment Volumes of Flocculated Kaolin Suspensions", Industrial and Engineering Chemistry, Vol.1, #1, pp.24-33.

Miller, R.J. and Law, P.F.(1963), "Threshold Gradient for Water Flow in Clay Systems", Soil Science Society of America, Proceedings, Vol.27, pp. 605-609.

Mitchell, J.K.(1955), "The Importance of Structure to the Engineering Behavior of Clay", PhD Dissertation, The University of California, Berkeley, 124 p.

Mitchell, J.K.(1960), "Components of Pore Water Pressure and Their Engineering Significance", Clays and Clay Minerals, Vol.9, pp. 162-184.

Mitchell, J.K., Hooper, D.R., and Campanella, R.G.(1965), "Permeability of Compacted Clay" Journal of the Soil Mechanics Division, ASCE, vol.91, # SM 4, pp.41-65.

Mitchell, J.K.(1973), "Influence of Mineralogy and Pore Solution Chemistry on the Swelling and Stability of Clays", in Proceedings of the Third International Conference on Expansive Clays, Haifa, Israel, pp. 11-24.

Mitchell, J.K. (1976), Fundamentals Of Soil Behaviour, John Wiley & Sons Inc., New York, 422 p.

Mitchell, J.K. (1986), "Practical Problems from Surprising Soil Behaviour", (The Twentieth Karl Terzaghi Lecture), Journal of the Geotechnical Engineering Division, ASCE, Vol.112, # 3, pp. 259-289.

Moell, C.(1977), Leachate and Gas Migration from the Clover Bar Sanitary Landfill - Edmonton, Alberta Environment, Earth Sciences Division, Report #1184.

Moell, C.E. (1980), Sanitary Landfill Studies in Alberta, Summary of Investigations to date, Report prepared for Alberta Environment, Research Management Division & Waste Management Branch, Edmonton, Alberta.

Montague, P.(1982), "Hazardous Waste Landfills: Some Lessons from New Jersey", Civil Engineering, ASCE, Sept. 1982, pp. 53-56.

Morrison, A.(1981), "Can Clay Liners Prevent Migration of Toxic Leachate", Civil Engineering, ASCE, July 1981, pp. 60-63.

Mott, C.J.B. (1981), "Anion and Ligand Exchange" in The Chemistry of Soil Processes, Eds. D.J. Greenland and M.H.B. Hayes, A Wiley-Interscience Publication, Toronto, pp. 179-220.

Mysels, K.J.(1967), Introduction to Colloid Chemistry, Interscience Publishers Inc., New York, 475 p.

Nagaraj, T.S. and Murthy, B.R.S.(1983), "Rationalization of Skempton's Compressibility Equation", Geotechnique, Vol.33, #4, pp. 433-443.

Nayak, N.V. and Christensen, R.W.(1971), "Swelling Characteristics of Compacted, Expansive Soils", Clays and Clay Minerals, Vol.19, pp. 251-261.

Neuzil, C.E. and Tracy, J.V.(1981), "Flow Through Fractures", Water Resources Research, Vol.17, #1, pp. 191-199.

Nickel, S.H.(1977), "A Rheoligical Approach to Dispersive Clays" IN Proceedings of the Symposium on Dispersive Clays, Related Piping and Erosion in Geotechnical Projects, ASTM #623, J.L. Sherard and R.S. Decker, Eds., Chicago, Ill., pp. 303-312.

Norrish, K.(1954), "The Swelling of Montmorillonite", Discussion, Faraday Society, Vol.18, pp. 120-134.

Norrish, K.(1972), "Forces Between Clay Particles", Proceedings of the International Clay Conference, Madrid, Spain, pp. 375-383.

Norrish, K., and Quirk, J.P.(1954), "Crystalline Swelling of Montmorillonite", Nature, vol.173, pp. 255-257.

Olson, H.W.(1960), "Hydraulic Flow Through Saturated Clays", Clays and Clay Minerals, Vol.9, pp. 131-161.

Olson, H.W.(1965), "Deviations From Darcy's Law in Saturated Clays", Soil Science Society of America, Proceedings, Vol.29, pp. 135-140.

Olson, R.E. and Mesri, G.(1970), "Mechanisms Controlling Compressibility of Clays", Journal of the Soil Mechanics Division, ASCE, Vol.96, SM6, pp. 1863-1878.

Olson, R.E., and Daniel, D.E.(1981), "Measurement of the Hydraulic Conductivity of Fine Grained Soils", IN Permeability and Groundwater Contaminant Transport, ASTM STP 746, T.F.Zimmie and C.O.Riggs, Eds., American Society for Testing Materials, pp. 18-64.

Pawluck, S. and Bayrock, L.A.(1969), Some Characteristics and Physical Properties of Alberta Tills, Bulletin #26, Research Council of Alberta, Edmonton, Alberta, 72 p.

Perloff, W.H., and Baron, W.(1976), Soil Mechanics Principles and Applications, The Ronald Press Co., New

York, 745 p.

Philip, J.R. and Knight, J.H.(1984), "Mechanics of Colloidal Suspensions with Application to Stress Transmission, Volume Change and Cracking in Clay Soils", Proceedings ISSS Symposium on Water and Solute Movement in Heavy Clay Soils, eds., J. Bouma and P.A.C. Raats, ILRI Publication 37, Wageningen, The Netherlands, pp. 39-44.

Plummer, P.S. and Gostin, V.A.(1981), "Shrinkage Cracks: Dessication or Synaerisis", Journal of Sedimentary Petrology, Vol.51, #4, pp. 1147-1156.

Pohland, F.G. and Engelbrecht, R.S.(1976), Impact of Sanitary Landfills - An Overview of Environmental Factors and Control Alternatives, Report prepart for American Paper Institute, 82 p.

Pusch, R.(1970), Clay Microstructure, Document D8: 1970, National Swedish Building Research, 76 p.

Quasim, S.R., and Burchinal, J.C.(1970), "Leaching of Pollutants from Refuse Beds", Journal of the Sanitary Engineering Division, ASCE, vol.96, SA1, pp.49

Quigley, R.M., Crooks, V.E. and Fernandez, F.(1984), "Engineered Clay Liners (An Overview Report)", Proceedings Working Seminar as Design and Construction of Municipal and Industrial Waste Disposal Facilities, Canadian Geotechnical Society and Consulting Engineers, Toronto, pp. 115-134.

Quirk, J.P.(1964), "The Nature and Strength of Interparticle Bonds in Natural and Stabilized Soils" In, Mechanisms of Soil Stabilization, Proceedings of a Colloquium held at Syndal, Victoria, pp. D3-1 - D3-8.

Quirk, J.P.(1968), "Particle Interaction and Soil Swelling", Isreal Journal of Chemistry, Vol.6, pp. 213-234.

Quirk, J.P. and Schofield, R.K.(1955), "The Effect of Electrolyte Concentration on Soil Permeability", Journal of Soil Science, Vol.6, pp. 163-178.

Ravina, I.(1972), "The Influence of Vegetation on Moisture and Volume Changes", Geotechnique, Vol.32, pp.151-157.

Reeve, R.E., Bower, C.A., Brooks, R.H., and Gschwend, F.B.(1954), "A Comparison of the Effects of Exchangeable Sodium and Potassium Upon the Physical Condition of Soil", Soil Science Society of America, Proceedings, Vol.18, pp 130-132.

Rich, J.L.(1951), "Three Critical Environments of Deposition and Criteria for Recognition of Rock Deposited in each Them", Geological Society of America Bulletin, Vol.62, pp. 1-20.

Ridley, K.J.D., and Bewtra, J.K.(1983), "Breakthrough of Brine Through Compacted Fine Grained Soils", Proceedings, 38th Annual Purdue Industrial Waste Conference, West Lafayette, Indiana.

Ridley, K.J.D.(1985), "The Impact of a NaCl Brine on the Behavior of a Compacted Fine Grained Soil-A Laboratory Study", Ph D Dissertation, University of Windsor, Ontario, 455 p.

Rolfe, P.F. and Alymore, L.A.G.(1977), "Water and Salt Flow Through Compacted Clays: I. Permeability of Compacted Illite and Montmorillonite", Soil Science Society of America, Journal, Vol.41, pp. 489-495.

Rosenqvist, I.Th.(1959), "Physico-Chemical Properties of Soils: Soil-Water Systems", Journal of the Soil Mechanics and Foundation Division, ASCE, Vol.85, SM 2, pp. 31-53.

Salas, J.A.J. and Serratosa, J.M.(1953), "Compressibility of Clays", Proceedings, Third International Conference on Soil Mechanics and Foundation Engineering, pp. 192-198.

Samarasinghe, A.M., Huang, Y.H., and Drnevich, V.P.(1982), "Permeability and Consolidation of Normally Consolidated Soils", Journal of the Geotechnical Engineering Division, ASCE, vol.108, # GT 6, pp. 835-849.

Sanks, R.L., La Plante, J.M., and Gloyna, E.F.(1975), Survey - Suitability of Clay Beds for Storage of Industrial Solid Waste, Technical Report, University of Texas at Austin.

Schofield, R.K.(1946), "Ionic Forces in Thick Films of Liquid Between Charged Surfaces", Transactions, Faraday Society, Vol.42B, pp. 219-227.

Schubert, W.R., Harrington, T.J. and Finno, R.J.(1984), "Glacial Clay Liners in Waste Disposal Practice", Proceedings, 1984 Specialty Conference, Environmental Engineering, ASCE, Los Angeles, California, pp. 36-41.

Scott, R.F.(1965), Principles of Soil Mechanics, Addison-Wesley, Landon, 550 p.

Seed, H.B., Mitchell, J.K., and Chan, C.K.(1962), "Swell and Swell Pressure Characteristics of Compacted Clay", Highway Research Board, Bulletin #13, pp. 12-39.

Sergeyev, Y.M., Grabowska-Olszewska, B., Osipov, V.I., Sokolov, V.N. and Kolomenski, Y.N. (1980), "The Classification of Microstructure of Clay Soils", Journal of Microscopy, Vol.120, pp. 237-260.

Shainberg, I. and Kemper, W.D.(1966), "Hydration Status of Adsorbed Cations", Soil Science Society America Proceedings, Vol.30, pp. 707-713.

Shainberg, I., and Kaiserman, A.(1969), "Kinetics of the Formation and Breakdown of Ca Montmorillonite Tactoids", Soil Science Society of America, Proceedings, Vol.33, pp.547-551.

Shainberg, I., Bresler, E., and Klausner, Y.(1971), "Studies on Na/Ca Montomorillonite Systems, I. The Swelling Pressure", Soil Science, Vol.111, #4, pp. 214-219.

Sherwood, P. T.(1979), "Effects of Sulfates on Cement and Lime-stabilized Soils", Highway Research Board, Bulletin #353, pp.98-107.

Sides, G. and Barden, L.(1971), "The Microstructure of Dispersed and Flocculated Samples of Kaolinite, Illite and Montmorillonite", Canadian Geotechnical Journal, Vol.8, pp. 391-399.

Singh, A.(1976), Soil Engineering - In Theory and Practice, Vol.1, Asia Publishing House, London, 814 p.

Sittig, Marshall (1979), Landfill Disposal of Hazardous Wastes and Sludges, Pollution Technology Review #62, Noyes Data Corporation, New Jersey, U.S.A.

Statton, C.T., and Mitchell, J.K.(1977), "Influence of Eroding Solution Composition on Dispersive Behaviour of a Compacted Clay Shale", IN Proceedings of the Symposium on Dispersive Clays, Related Piping, and Erosion in Geotechnical Projects, ASTM #623, J.L. Sherard and R.S. Decker, Eds., Chicago, Ill. pp. 398-407.

Sridharan, A. and Venkatappa Rao, G.(1971), "Effective Stress Theory of Shrinkage Phenomena", Canadian Geotechnical Journal, Vol.8, #4, pp. 503-513.

Sridharan, A. and Venkatappa Rao, G.(1973), "Mechanisms Controlling Volume Change of Saturated Clays and the Effective Stress Concept", Geotechnique, Vol.23, #3, pp. 359-382.

Sridharan, A. and Venkatappa Rao, G.(1979), "Shear Strength Behavior of Saturated Clays and the Role of Effective Stress Concept", Geotechnique, Vol.29, #2, pp. 177-193.

Sridharan, A. and Jayadeva, M.S.(1982), "Double Layer Theory and Compressibility of Clays", Geotechnique, Vol.32, #2, pp. 133-193.

Stumm, W., and Morgan, J.J.(1970), Aquatic Chemistry: An Intrduction Emphasizing Chemical Equilibria in Natural Waters, Wiley-Interscience, New York.

Stumm, W. and Morgan, J.J.(1981), Aquatic Chemistry, Second Edition, Wiley-Interscience, New York.

Styron, C.R. and Fry, Z.B. (1979), The Gas Cleaning Sludge Leachate/Liner Compatibility Investigation Interim Report, EPA-600/2-79-136, US EPA, Cincinnati, Ohio, USA

Subba Rao, K.S. and Satyadas, G.C.(1986), "Pattern of Volume Change Development", Journal of the Geotechnical Engineering Division, ASCE, Vol.112, #2, pp. 203-206.

Talibudeen, O.(1981), "Cation Exchange in Soils" in The Chemistry of Soil Processes, Eds. D.J. Greenland, M.H.B. Hayes, A Wiley-Interscience Publication, pp. 115-178.

Taylor, A.W.(1959), "Physico-Chemical Properties of Soils: Ion Exchange Phenomena", Journal of the Soil Mechanics and Foundations Division, ASCE, Vol.85, #SM2, pp. 19-29.

Taylor, D.A.(1934), "Detailed Stratigraphy of Edmonton District", MSc Thesis, Department of Geology, The University of Alberta.

Taylor, D.W.(1942), Research on Consolidation of Clays, Serial #82 Massachusetts Institute of Technology, Department of Civil Engineering, Cambridge, 147 p.

Taylor, D.W.(1948), Fundamentals of Soil Mechanics, Wiley, New York.

Theng, B.K.G.(1974), The Chemistry of Clay-Organic Reactions, Halsted Press, John Wiley & Sons, Inc., New York.

Thomas, G.W. and Phillips, R.E.(1979), "Consequences of Water Movement in Macropores", Journal of Environmental Quality, Vol.2, pp. 149-153.

Tovey, N.K. and Wong, K.Y.(1980), "The Microfabric of Deformed Kaolin", Journal of Microscopy, Vol.120, pp. 329-342.

Turk, L.J.(1974), "Leakage from Solar Evaporation Ponds Associated with Sediment-Brine Interactions", Fourth International Symposium on Salt, Northern Ohio Geological Society, pp. 403-406.

Umehara, Y. and Zen, K.(1982), "Consolidation Characteristics of Dredged Marine Bottom Sediments with High Water Content: *Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering*, Vol.22, #2, pp.

van Olphen, H.(1963), *An Introduction to Clay Colloid Chemistry*, A Wiley Interscience Publication, New York, 301 p.

Verway, E.J.W. and Overbeek, J.Th.G.(1946), "Long Distance Forces Acting Between Colloidal Particles", *Transactions, Faraday Society*, Vol.42B, pp. 117-131.

Waldran, L.J. and Constantin, G.K.(1967), "Bulk Volume and Hydraulic Conductivity Changes During Sodium Saturation Tests", *Soil Science Society of America, Proceedings*, Vol.32, pp. 175-179.

Waldron, L.J. and Constantin, G.K.(1968), "Bulk Volume and Hydraulic Conductivity Changes During Sodium Saturation Tests", *Soil Science Society of America, Proceedings*, Vol.32, pp. 175-179.

Wang, M.C. and Huang, C.C.(1984), "Soil Compaction and Permeability Prediction Models", *Journal of the Environmental Engineering, ASCE*, Vol.110, #6, pp. 1063-1083.

Warkentin, B.P., Bolt, G.H., and Miller, R.D.(1957), "Swelling Pressure of Montmorillonite", *Soil Science Society of America, Proceedings*, vol.22, pp. 495-497.

Weiss, Samuel.(1974), *Sanitary Landfill Technology*, Pollution Technology Review #10, Noyes Data Corporation, New Jersy.

Weidelich, W.C.(1958), "Influence of Liquid and Clay Mineral Type on Consolidation of Clay-Liquid Systems", *Water and its Conductance in Soils*, Special Report #40, Highway Research Board.

White, W.A.(1961), "Colloidal Phenomena in Sedimentation of Argillaceous Rocks", *Journal of Sedimentary Petrology*, Vol.31, pp. 560-565.

Witherspoon, P.A., Amick, C.H., Gale, J.E. and Iwai, K.(1979), "Observations of a Potential Size Effect in Experimental Determination of the Hydraulic Properties of Fractures", *Water Resources Research*, Vol.15, #5, pp. 1142-1146.

Yanful, E.K.(1982), "Geotechnical Properties of Clay Soils in a Brine Environment", MSc Thesis, University of Windsor, Ontario, 221 p.

Yong, R.N., Sheeran, D.E. and Nagaraj, T.S.(1975), "Soil Composition Control of Fabric and Response Behavior", Proceedings of the International Clay Conference, pp. 663-672.

Yong, R.N. and Warkentin, B.P.(1969) "Flow of Water In Partially-Saturated Expansive Soils" Proceedings, Second International Research & Engineering Conference on Expansive Soils, pp. 85-98.

Yong, R.N. and Warkentin, B.P.(1975), Soil Properties and Behavior, Geotechnical Engineering 5, Elsevier Scientific Publication Co., N.Y, 449 p.

APPENDIX A - Notations

ρ = space distribution of charge and also mass density of a permeant,

x_0 = distance from surface,

ϵ = dielectric constant,

Z = valence,

C = concentration,

ψ = electric potential,

ϕ = variable related to the central concentration,

n_i = concentration of ions,

e = unit electronic charge (16.0×10^{-20} coulomb or 4.8×10^{-10} esu), also void ratio

T = absolute temperature (in °K),

δ = double layer thickness,

K_d = Debye Quantity,

n_0 = concentration of ions, a large distance away from the surface,

k = Boltzmann constant,

β = a constant, dependent upon the temperature $\simeq 10^{15}$ cm/mmoles,

Γ = surface density of charge, in milliequivalent/cm^2,

C_d = cation concentration in the central plane,

C_0 = concentration in the bulk liquid pressed out from the system or large distance away from particle surface,

R = gas constant,

d = half distance between parallel clay platelets,

G_s = specific gravity of soil particles,

y_c = non-dimensional midplane potential function,

x_0 = distance between clay surface and imaginary plane at which potential becomes infinity,

y = non-dimensional potential at distance n from clay surface,

N = Avagadro's number,

γ_w = density of water = 1.0 g/cm^3,

S = specific surface of soil solids,

K = intrinsic permeability,

k = hydraulic conductivity,

d_e = effective grain size, d_{10},

q = flow rate,

i = hydraulic gradient,

A = x-sectional area of liquid flow,

T_v = Terzaghi's time factor,

m_v = Coefficient of volume compressibility,

APPENDIX B - Clay Soils

APPENDIX B1

CLAY MINERALOGY AND COMPOSITION

In general, the term clay implies a natural, earthy, fine grained material which develops plasticity when mixed with a limited amount of water. However, the term "clay", as used in soil mechanics has two meanings. It refers to all constituents of a soil smaller than 0.002 mm (or 2 microns). Most clay sized particles in nature are clay minerals, although this fraction can also contain other minerals such as quartz and feldspar.

Sands and silts are relatively inert to solutions passing through the pore spaces around them, whereas, clay minerals interact with those permeating solutions. The amount and type of clay minerals are the major factors controlling the volume change behavior of a soil and therefore its bulk permeability.

Clay minerals are very fine grained, crystalline hydroaluminosilicates. Chemical analysis of clays show them to be composed essentially of silica, alumina and water, frequently with appreciable quantities of magnesium or iron, alkalis (sodium or potassium) and alkaline earth (for example, calcium) metals (Grim, 1968).

The aluminum - oxygen combinations, are the basic structural units, which are bonded together in such a way that sheets are formed. Common clay minerals consist of two fundamental building blocks or sheets: a silica tetrahedral sheet, and an alumino octahedral sheet. The different clay minerals are the result of variations in stacking of these sheets into layers, the bonding between layers, and the substitution of other ions for the original ions, aluminum and silica.

The most common clay minerals found in soils are kaolinites, chlorites, illites (clay mica), montmorillonites and vermiculites. Most clay soils contain mixtures of these clay minerals, but there are many that are substantially monomineral. However, certain clay minerals (eg. montmorillonite), even in small quantities, exert a tremendous influence on the attributes of a clay material.

Basic Units Forming Clay Minerals

The two basic units in a clay mineral structures are the silicon tetrahedron and the aluminum or magnesium octahedron.

The tetrahedral unit cell consists of a silicon ion surrounded by four oxygen ions. These tetrahedra are bonded together in an open hexagonal pattern, with each of the oxygen ions in the base common to another unit. This bonding produces a sheet structure of silicon ions between two layers of oxygen ions.

The octahedral unit cell consists of six hydroxyl ions surrounding an aluminum or magnesium ion. These octahedral units are bonded together in a hexagonal arrangement, with each hydroxyl ion common to three octahedral units. This sheet consists of a planar layer of aluminum or magnesium ions between two planar layers of hydroxyl ions.

These molecular sheets are bonded together into two or three layer units, to form different clay minerals.

Clay Minerals

Major differences in clay mineral composition is due to the difference in type and number of sheets, the amount of ion substitution and the type and amount of ions between sheets.

The kaolinite crystal consists of repeating layers, each layer consisting of a silica sheet and an alumina sheet sharing a layer of oxygen atoms between them. The combined silica-alumina sheets are held together by hydrogen bonding between hydroxyls, from the alumina sheet on one face and oxygens from the silica sheet on the opposite face of the layer. These forces are relatively strong, preventing hydration between layers and allowing many layers to build up. Therefore, the liquid limit and the activity of kaolinites are fairly low.

The repeating layers of the chlorite minerals are composed of a silica sheet, an alumina sheet, a second silica sheet and another alumina or a brucite sheet ($Mg(OH)_2$). Substitution of other cations for silicon or aluminum occurs in chlorite. The chlorites found in soils may have a partly random stacking of layers and some hydration between layers.

Illites have repeating layers of an alumina sheet between two silica sheets with oxygen atoms shared to give a unit (3 layer system) which is four oxygen atoms thick. The layers are held together by potassium ions, which are the correct size to fit the hexagonal holes of the silica sheet. These potassium ions are non-exchangeable. Substitution of aluminum ions for silicon provides negative charges which balance the positive potassium ions. Soils containing mineral illite have properties intermediate between kaolinite, with a low activity, and montmorillonite with high activity.

Montmorillonite has a structure similar to that of illite, except that isomorphous substitution occurs mainly in the alumina sheet, with Mg^{+2} or Fe^{+2} substituting for Al^{+3} in the dioctahedral minerals (the term "dioctahedral" is used for clay minerals in which only two thirds of the octahedral positions are filled). The space between the combined sheets is occupied by water molecules and exchangeable cations, instead of potassium ions. The bonds are weaker than for illite, with the result that montmorillonites can swell much more in the presence of water. Montmorillonite has a higher SiO_2 and lower Al_2O_3 content than kaolinite, because it contains an extra tetrahedral sheet. Montmorillonite, being able to swell to very high volumes, exhibits a higher liquid limit and a high activity.

APPENDIX B2

CLAY SOILS IN ALBERTA

GLACIAL TILL

"Glacial Till" is the most significant parent material from which most of the Alberta soils are derived. It has been reported that till deposits comprise approximately 70% of the surficial deposits in Alberta (Pawluck and Bayrock, 1969).

By definition, glacial till is a material deposited directly from glaciers without washing or sorting. It is an unsorted admixture of local bedrock and of foreign rock material from a distant source. But the bulk till material is derived locally, reflecting composition of underlying or nearby strata (Bayrock and Hughes, 1962).

Mechanical Composition of Alberta Till

Bayrock and Hughes (1962) reported the following average mechanical composition for Alberta tills, which is common to central Alberta:

Sand - 41.1%
Silt - 31.4%
Clay - 27.5% (maximum = 35%).

However, they noted the possible existence of marked local deviations.

Particle size distribution of tills are known to vary markedly from one region to another. In Northeastern Alberta sand content (60-70%) is higher, and clay content (10-20%) is lower than average. However, tills in Northwestern

Alberta contains very high clay percentages, sometimes greater than 70%. In general, tills in Southern Alberta are less clayey and more sandy and/or silty than those of Eastern Alberta, which is less clayey than those of the North-Central and Northwestern Alberta.

Clay Minerals in Found in Glacial Tills

The clay fraction of Alberta tills is comprised of variable amounts of montmorillonite, illite, kaolinite, chlorite and fine grained quartz. The approximate composition of the clay sized fraction is as follows:

- montmorillonite - 20-60%,
- illite - 15-48%,
- chlorite - 10-20%,
- kaolinite - 10-20%,
- silt - 30-70%.

Montmorillonite is the dominant clay mineral in central and southern regions, whereas illites dominate the clay content in nothern regions. Major clay mineral in tills of eastern Alberta is montmorillonite. Lesser amounts of illite and kaolinite are also found. In tills of area just east of Edmonton, in northwestern Alberta, and in the Fort McMurray area, major clay mineral is montmorillonite, followed by illite and kaolinite (a similar composition is reported for lake sediments found in these areas). However, till in Calgary area (known as Cordilleran Till) exhibit a lower montmorillonite to illite ratio (Gravenor and Bayrock, 1962).

Index Properties of Alberta Tills

Liquid Limit of Alberta tills range from a low 15% to a medium 50% (general range 25-35%). Majority of Alberta tills are ranked as low to medium plastic soils, and when considered with clay content, establish the tills to be relatively inactive to normally active (Pawluck and Bayrock, 1969).

Tills in the north exhibit plasticity indices in the range from 4 to 28% and liquid limit (LL) between 18 to 39%. Plasticity Index (PI) of southern and central tills range between 14-20% and LL between 22-48%. However, wide variations of these values, due to the variability of soluble salt content of till, has been reported (Taylor, 1934). Tills in Southern Alberta, containing larger amounts of montmorillonite, exhibit an average activity value (rather than a high activity value), due to high soluble salt content and a preponderance of coarse grained particles.

Sulfate is the major anion found in Alberta tills. In southern and central Alberta soils, the principal salt is Na_2SO_4. $MgSO_4$ has also been detected in some samples. $MgSO_4$ is the dominant soluble salt in the Peace River region. However, Pawluck and Bayrock (1969) noted that the dominant exchangeable cation in montmorillonites found in glacial tills to be Ca^{+2}, whereas montmorillonite in shales to be Na saturated.

Soils derived from tills are known to exhibit low pH values, high lime content and moderate to high salinity.

LACUSTRINE DEPOSITS

Lacustrine outwash and aeolian materials are derived from till through sorting of clayey and sandy constituents and therefore range from pure sand to pure clay. Lake deposits occur mostly in the northern part of the province. Lake deposits in central and northwestern Alberta are composed largely of silt and clay. In northeastern Alberta and parts of central and southern Alberta the lake deposits consist largely of sand.

Lake Edmonton, at one time, covered most of the Edmonton district and extended far to the west. Only the very eastern part of the district and a small area in the northwest were not covered by Lake Edmonton.

As noted by Bayrock and Berg (1966), glacial sediments in Edmonton area consists of silts and clays, with scattered pebbles, laid down in a large preglacial lake at the close of the Wisconsin glacial period. They are more clayey in the uppermost few feet than in the lower portions. Mechanical analysis of typical lake deposits from the lower beds in the city of Edmonton area averaged 5% sand, 40% silt and 55% clay (Bayrock and Berg, 1966). The proportion of montmorillonite in the clay fraction is high and Ca-saturated. Average liquid limit was 65% (as opposed to 40% for the till found in city of Edmonton Area) and the plasticity index (PI) was 40%.

SOILS IN EDMONTON DISTRICT

Bedrock of the Edmonton area is composed of shales and sandstones belonging to 3 formations; Edmonton, Bearpaw and Belly River. The Edmonton formation, containing large proportions of clay and bentonite, underlies most of the Edmonton district (Taylor, 1934).

Edmonton formation predominantly consists of soft sandstones, shales and coal seams. The shales are soft and often graded as hard or soft clays. These show high swelling potential, Na being the dominant cation present in the exchange complex. Some of the beds are merely clays containing bentonite (high Na montmorillonite content). Bentonites are also known to occur in bands.

Buff colored till or boulder clay or grey or light buff colored lake deposits, overlay the Edmonton formation. In the Edmonton area, till makes up most of the ground moraine (a large area lies east of the city of Edmonton, and in the northwestern portion of the Edmonton district) and underlies most of the other glacial deposits of the area (Bayrock and Hughes, 1962).

For an example, Millwood area is covered by materials derived from the lacustrine clays, which is underlain by glacial till (Lindsay and Scheelar, 1972). A typical x-section consists of:

- 12 feet - lacustrine clay,
- >10 feet - clay loam till,
- bentonitic shales and sandstone of the Edmonton

formation.

The lacustrine clay material found in Millwoods area gernerally consists of 38.65% clay sized particles. The liquid limit ranges between 43 and 64%, plasticity index ranging from 21 to 39%. Textural classification is either CL or CH. However local variations are known to occur (Lindsay and Scheelar, 1972). In the western portion of Millwoods, seven feet of highly plastic lacustrine clay, fitting CH in the unified system, could be found on the surface. These soils have a high shrink/swell potential and a moderate acitivity (0.5 to 0.9).

APPENDIX C - Fabric of Clay Soils

Soil Fabric is considered to be made up of the orientation of particles and the texture of the material. Particle orientation refers to the positions of adjacent particles relative to each other, whereas texture refers to the particles and groups of particles within the soil mass.

Lambe (1958a), Mitchell (1960) proposed two basic premises of particle orientation; a dispersed system consisting of particles in a parallel arrangement and a flocculated system consisting of particles in a random arrangement. However recent scanning microscopic studies (Sides and Barden, 1971) revealed that these concepts of flocculated and dispersed structures to be too simple, particularly the concept of flocculated structure.

van Olphen (1963), applying the double-layer interactions, stated that three different modes of particle association may occur when a suspension of plate-like clay particles flocculates: edge to edge (EE), edge to face (EF) and face to face (FF). van Olphen summarized comprehensively the various modes of particle association in clay suspensions and proposed the terms flocculated, dispersed and aggregated to designate the microstructures. The proposed clay particle arrangements are duscussed in detail by several authors (Sergeyev, et al. 1980; Collins and McGowan, 1974; Sides and Barden, 1971; Pusch, 1970).

The most commonly used term in soil mechanics, "flocculated" refers to a soil in which the net interparticle force between adjacent particles during deposition was attraction (Lambe, 1958a). This requires a contracted double layer. Flocculation implies the essentially open-structured EE and EF arrangements. These arrangements produce a "cardhouse" (single particle arrangement) or more likely the "bookhouse" (sometimes referred to as "salt flocculated") structure. As noted by van Olphen (1963), EE and EF association leads to a three-dimensional voluminous open structure. An even more open-flocculated or "honeycombe" structure is also reported in literature (Sides and Barden, 1971).

According to Lambe (1958a), the "cardhouse" structure is formed when clays are sedimented in fresh water, whereas, clay soils sedimented in salt water produces the "bookhouse" microstructure.

The term "dispersed" refers to a soil in which the net interparticle force between adjacent particles during deposition was repulsion (Lambe, 1958a). This requires a large or expanded double layer. Thus dispersed implies a rather close-packed structure involving essentially FF arrangements (Sides and Barden, 1971).

However, a more realistic representation of a dispersed structure is the "turbostratic" microstructure proposed by Aylmore and Quirk (1962). A number of investigators have described the existence of a microstructure formed by accumulations of axially oriented particles. The groups or aggregates, in which the particles are approximately parallel to each other are called "domains" or "tactoids" (Pusch, 1970). When the domains form a random array, the sample is isotropic at the macroscale, although anistropic at the microscale. A high degree of domain orientation is a limiting case of turbostratic structure and has been called a "complete preferred orientation" by Smart (1969).

Thus "turbostratic" implies large domains with a gradual change in orientation, with the boundary between domains often diffuse and indistinct. In such a close packed arrangement, no large interdomain macrovoids or cavities exist. "Salt flocculated" implies smaller domains, often containing few particles in a domain or a tactoid. The boundary between domains is usually well defined. The overall arrangement is a random and open structure, dominated by large interdomain macrovoids and cavities.

The formation of domains is enhanced when calcium is the principal exchangeable cation and/or prevalent cation in a sedimenting environment. Furthermore, when the presence of domain-like aggregates, deposited as such, are available for nucleation of further materials, domain formation is considerably increased (Pusch, 1970).

The various observations reported in literature point to the fact that the nature of particle interaction and orientation to be very complex. While there is some evidence of the occurrence of "cardhouse" microfabrics, they are relatively rare in natural clay soils (Marsland et al. 1982). Collins and McGowan (1974) reported that single plate arrangements of any form are rare in nature, whereas groups of clay plates interacting in various forms are common.

There seems to be a wide range of macrofabric features present in a clay soil system. Many of the elementary particle arrangements described can be found to exist side by side in any one soil (Collins and McGowan, 1974). Nevertheless, it has been found that certain clay particle arrangements were more predominant in certain types of soil. Bookhouse and stepped flocculated (an arrangement of EE, FF and EF contacts) arrangements found in marine soils, dispersed or turbostratic group arrangements seen in brackish water soils, are a few examples.

Lambe (1958a) contended that sedimentation in a high electrolyte concentration (as in the case with marine clays) leads to flocculation, with a higher degree of particle parallelism leading to the "stepped flocculated" arrangement. Lambe considered that sedimentation in intermediate electrolyte concentrations (such as found in brackish water) would lead to dispersion and FF particle arrangement (turbostratic arrangements). Although, these structures seem too simple to explain completely the nature of particle orientation in a natural soil or even in a laboratory sedimented soil, a fairly good explanation of a sediment behavior could be found by considering a soil sediment to contain an array of similar arrangements side by side. This approach was amply demonstrated by Collins and McGowan (1974) in their study of microfabric features in natural soils.

Definitions of some of the terms used in describing clay soil microfabric features, as adopted from Collins and McGowan (1974) are given below:

- Macrofabric - the level of fabric which can be observed with the naked eye or hand lens,
- Microfabric - the level of fabric which can only be observed with the aid of the polarizing light microscope,
- Cardhouse - EF interaction of clay plates forming an open fabric,
- Non salt flocculation - an open fabric formed by EF or EE interaction of clay plates resulting from sedimentation in salt free water,
- Salt flocculation - an open fabric formed by the EF, EE and EF interaction of clay plates from sedimentation in very high salt concentrations,
- Dispersion - an oriented fabric formed essentially by FF interactions resulting from sedimenting in medium salt, brackish water (Lambe, 1958a),
- Dispersed - van Olphen's (1963) description of an arrangement in which no FF association between particles exist,
- Aggregated - van Olphen's (1963) description of FF association of several particles,
- Flocculated - van Olphen's (1963) description of EF or EE association of particle,

- Domain or tactoid - a fabric unit consisting of parallel oriented clay plates,
- Turbostratic groups - the turbulent array of fabric units consisting of parallel oriented clay plates,
- Bookhouse - EF interaction of clay plate packets forming an open fabric which is analogous to the cardhouse fabric,
- Aggregation - a fabric unit within which there is interaction between particles of various sizes (Pusch, 1970),
- Cluster - a grouping of particle domains into a larger fabric unit,
- Ped - an individual soil aggregate consisting of a cluster of primary particles, and separated from adjoinging peds by surface of weakness or a grouping of clusters into a larger fabric unit,

- Intra-pedal pores - pores occuring within peds,
- Inter-pedal pores - pores occuring between peds,
- Transpedal pores - pores transversing the soil material without any specific relationship to the occurrence of peds, extending beyond the limits of a single ped.

APPENDIX D - Double Layer Equations

APPENDIX D1

THEORETICAL DISTRIBUTION OF EXCHANGEABLE IONS AROUND A SINGLE CLAY PARTICLE

Cations are attracted toward, and anions are repelled from, negatively charged soil colloids. Such interactions follow Coulomb's law. Therefore, the strength of ion retention or repulsion increases with increasing ion charge, with increasing colloid charge, and with decreasing distance between the colloid surface and the ions.

The increased cation concentration in the diffuse double layer develops a counter tendency for cation diffusion away from the surface, in order to equalize, ultimately, the cation concentrations throughout the liquid phase.

The Coulomb electrical attraction and thermal diffusion is combined in the Boltzmann equation for the distribution of ions in the electric field, and is given by:

$$\frac{n_i}{n_{io}} = \exp \frac{- Z_i e \psi}{k T} \qquad \text{D1}$$

Where, C_i is related to n_i by the following equation:

$$C_i \{\text{in } \frac{\text{mmole}}{\text{cm}^3}\} = C_i \{ \text{ in } 1000 \frac{\text{mole}}{1}\} = \frac{1000\, n_i}{N} \qquad \text{D2}$$

and, N is the Avagadro's number

To completely understand the double layer phenomenon, change of electric potential with distance from the colloid surface should be estimated. The standard approach is to regard soil colloids as having constant surface charge but variable surface potential.

The distribution of potential varies with bulk solution salt concentration and with average valence of the counter ions, but the charge of the soil (CEC) remains constant.

The relation between the potential ψ, distance from the surface x, and volume concentration of charges or ions ρ is given by the Poisson equation:

$$\frac{d^2\psi}{dx^2} = \frac{-4\pi\rho}{\epsilon} \qquad \text{D3}$$

Where, x is the distance from particle surface.

By combining Boltzmann and Poisson equations, one can obtain the differential equation of the electric double layer:

$$\frac{d^2\psi}{dx^2} = \frac{-4\pi e}{\epsilon} \Sigma Z_i n_{io} \exp\left\{\frac{-Z_i e \psi}{k T}\right\} \qquad \text{D4}$$

The charge density (ρ) is contributed by the ions, so that the following equation is satisfied:

$$\rho = e \Sigma Z_i n_i \qquad \text{D5}$$

For the case of a single cation and anion species of equal valence, D4 reduces to:

$$\frac{d^2\psi}{dx^2} = \frac{8\pi n_o Z e}{\epsilon} \sinh\left\{\frac{Z e \psi}{k T}\right\} \qquad \text{D6}$$

Verway and Overbeek (1948), Bolt (1955) and van Olphen (1963) have proposed solutions for the above differentiation equation. Equation D6 can be used to estimate a "double layer thickness" ρ as follows:

When the surface potential is small, equation D6 can be approximated to,

$$\frac{d^2\psi}{dx^2} = \frac{8 \pi n_o Z e}{\epsilon} \left\{\frac{Z e \psi}{k T}\right\} = K^2\psi \qquad D7$$

where,

$$\psi = \psi_o e^{-kx} \qquad D8$$

and,

$$K = \left\{\frac{8 \pi n_o}{\epsilon k T}\right\}^{-1/2} e Z \qquad D9$$

In this case, the center of gravity of the diffuse charge is located at a distance, $x = 1/K_d$ from the surface, and this quantity $1/k_d$ is known as the "thickness of the double layer (δ)":

$$\delta = \frac{1}{K_d} = \frac{1}{e Z} \left\{\frac{8 \pi n_o}{\epsilon k T}\right\}^{-1/2} \qquad D10$$

K_d is known as the Debye quantity.

APPENDIX D2

THEORETICAL CONSIDERATIONS FOR INTERACTING DOUBLE LAYERS

The mathematical description of the double layer is usually generalized by introducing dimensionless parameters:

For the electric potential,

$$y = \frac{Z\ e\ \psi}{k\ T} \qquad \text{D11}$$

For the distance,

$$\Delta = K_d\ x = \left\{\frac{8\ \pi\ n_o\ e^2\ Z^2}{\epsilon\ k\ T}\right\}^{1/2} x \qquad \text{D12}$$

Now, from equation D11,

$$\frac{d^2y}{dx^2} = \frac{Z\ e}{k\ T}\ \frac{d^2\psi}{dx^2} \qquad \text{D13}$$

Substituting D11 and D13 in equation D5,

$$\frac{d^2y}{dx^2} = (K_d)^2 \sinh y \qquad \text{D14}$$

Therefore,

$$\frac{d^2y}{dx^2} = \frac{(K_d)^2}{2}\ (e^y - e^{-y}) \qquad \text{D15}$$

Y_c, the potential function at the midplane (ie. at the boundary conditions, $x = K_d d$, $y = x$, and $dy/dx = 0$) and subsequently the mid point concentration C_d, can be determined using one of the following techniques:

- using the elliptic integral, proposed by Bolt (1956),
- midplane potential taken as the sum of the double layer potentials at distance d based on the solution

for a single plate: ie. $Y_c = 2Y_c$, (van Olphen, 1963),

- using the Langmuir solution, as outlined by Yong and Warkentin (1975).

A relation between, the distance between the particles and the concentration of cations in the central plane between the particles, have been proposed by Verway and Overbeek (1948), Bolt (1955).

$$Z\sqrt{(\beta C_0)}(x_0 + d) = 2\sqrt{\frac{C_0}{C_d}} \int \frac{d\phi}{\sqrt{\{1 - (C_0/C_d)^2 \sin^2\phi\}}} \qquad \text{D16}$$

Where,

$x_o \cong 4/Z \beta T$

($\cong$ 1/Z Ang. for Illite, 2/Z Ang. for kaolinite, 4/Z Angstroms for montmorillonite).

ϕ is a variable, the actual value of which is immaterial for the evaluation of equation D16.

Equation D16 is valid only if the anions present in the system have the same valency as the cations. However, the deviations occuring if this is not so are very small.

Values for the complete elliptic integral are given in standard tables.

On the other hand, simillar results can be obtained by using the tabulations and the equations given by van Olphen (1963).

The midplane potential (from Y_c) is found from:

$$\int (2 \cosh y - 2 \cosh y_c)^{1/2} \, dy = -K_d \, d \qquad \text{D17}$$

The integral in D17 is evaluated using tables provided by van Olphen (1963) to give Y_c for given values of ϕ_o, d and K_d.

Thereafter, Boltzmann equation can be used to obtain the midplane concentrations:

$$n_c = n_o \exp (Y_c) \text{ and}$$

$$C_d = C_o \exp (Y_c) \qquad \text{D18}$$

This procedure holds true when $Y_c = 2Y_d$, or for small interactions or large plate separations (large $k_d d$), high C, high Z or small ϕ_o.

Langmuir approximation

If we neglect the anion term, the differential equation becomes:

$$\frac{d^2y}{dx^2} = (K_d)^2 \frac{e^y}{2} \qquad \text{D19}$$

First integration, with the boudary conditions $Y = Y_c$ at the midplane, where $dy/dx = 0$, gives:

$$\frac{dy}{dx} = -K_d \sqrt{e^y - e^{y_c}} \qquad \text{D20}$$

The second integration, with boundary conditions,

$y \rightarrow \infty$ at $n = 0$ produces

$$y_c = 2 \ln \frac{\pi}{K_d\, n} \qquad \text{D21}$$

Solving B17 for Y_c and equating with B20 gives,

$$\ln \frac{C_d}{C_o} = 2 \ln \frac{\pi}{K_d\, x} \qquad \text{D22}$$

By taking $n = d + x_o$ at the mid point and rearranging

$$C_d = \frac{\pi^2 C_o}{K^2 (x_o + d)^2} \qquad D23$$

However by substituting,

$$C_o = \frac{1000 n_o}{N} \text{ and } K^2 = \frac{8 \pi n_o e^2 Z^2}{\epsilon k T},$$

in equation D23, we get,

$$C_d = \frac{\pi^2}{Z^2 (d + x_o)^2} \frac{1000 \epsilon k T}{8 \pi e^2 N} \qquad D24$$

Bolt (1955) defined,

$$\beta = \frac{8 \pi e^2 N}{1000 \epsilon k T}$$

Therefore,

$$C_d = \frac{\pi^2}{Z^2 (d + x_o)^2 \beta} . \qquad D25$$

(Note : This equation can also be obtained from equation D15, as follows:

When C_o is small ($C_d >>> C_o$) and D16 reduces to:

$$Z \sqrt{\beta C_o} (x_o + d) = 2 \sqrt{C_o/C_d} \int d\psi \qquad D26$$

By integration for the limits, $\psi = 0$ and $\psi = \pi/2$ and rearranging, Eq. D25 can be obtained).

APPENDIX E - Time-Settlement Curves

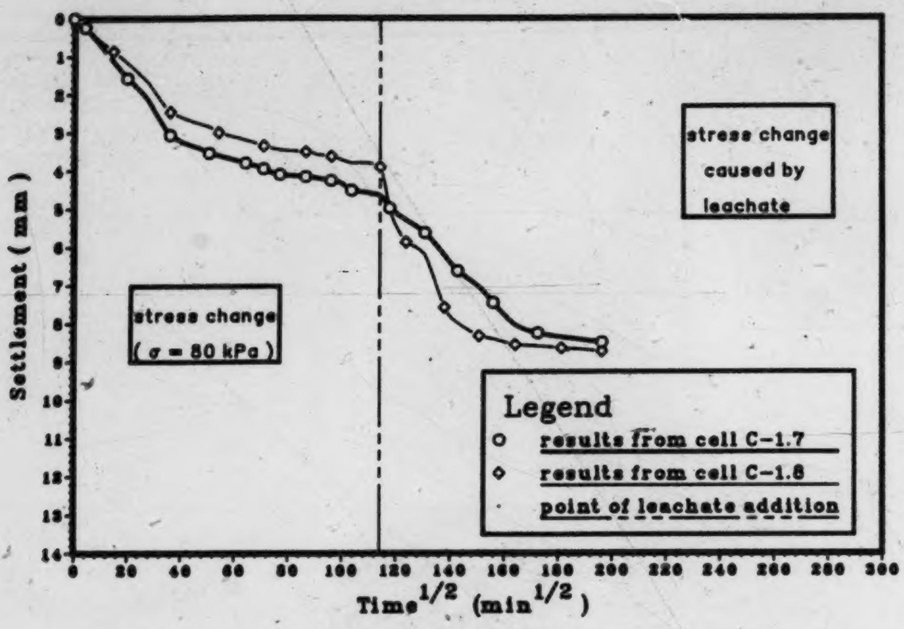

Figure E-1.1a Effect of Leachate Addition at 80 kPa

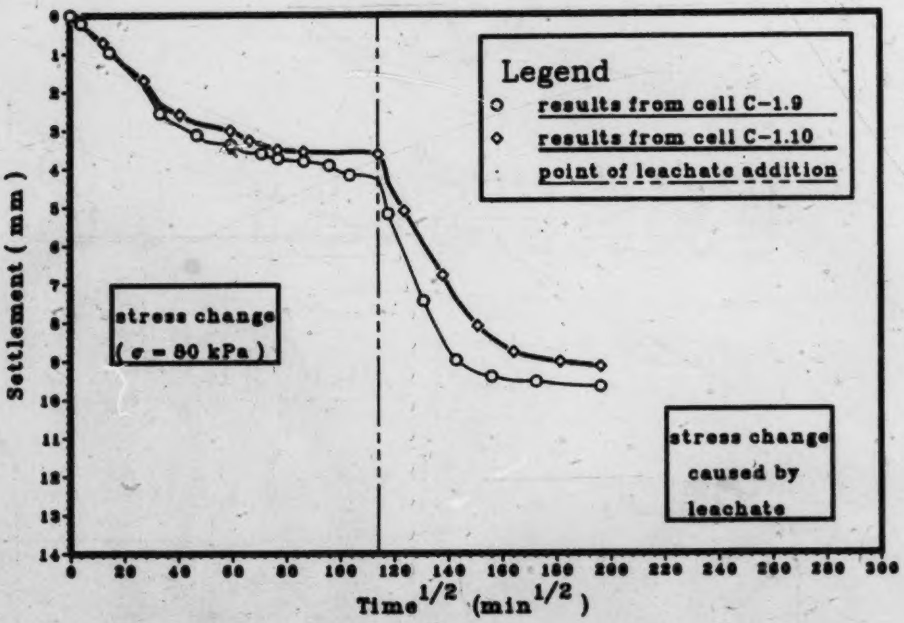

Figure E-1.2a Effect of Leachate Addition at 80 kPa

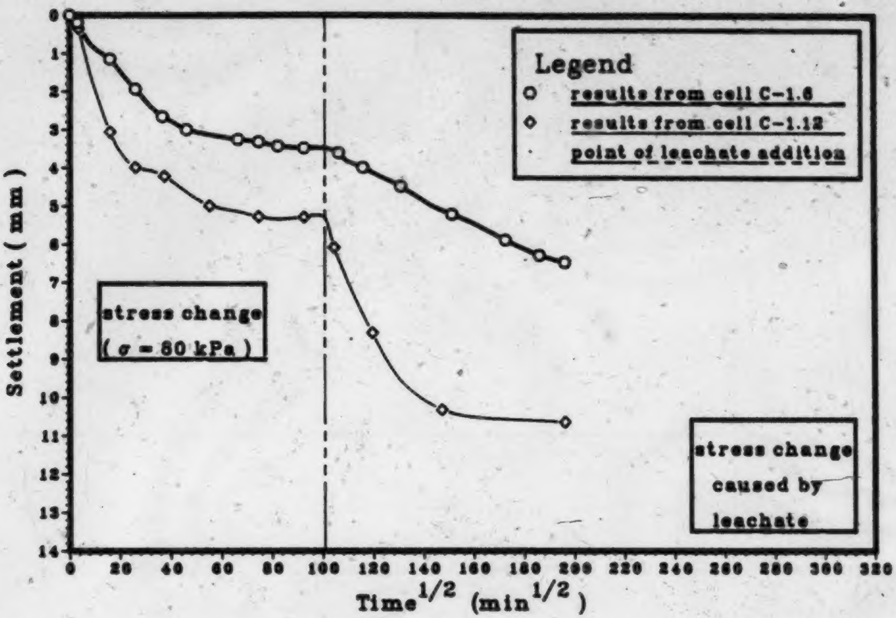

Figure E-1.3a Effect of Leachate Addition at 80 kPa

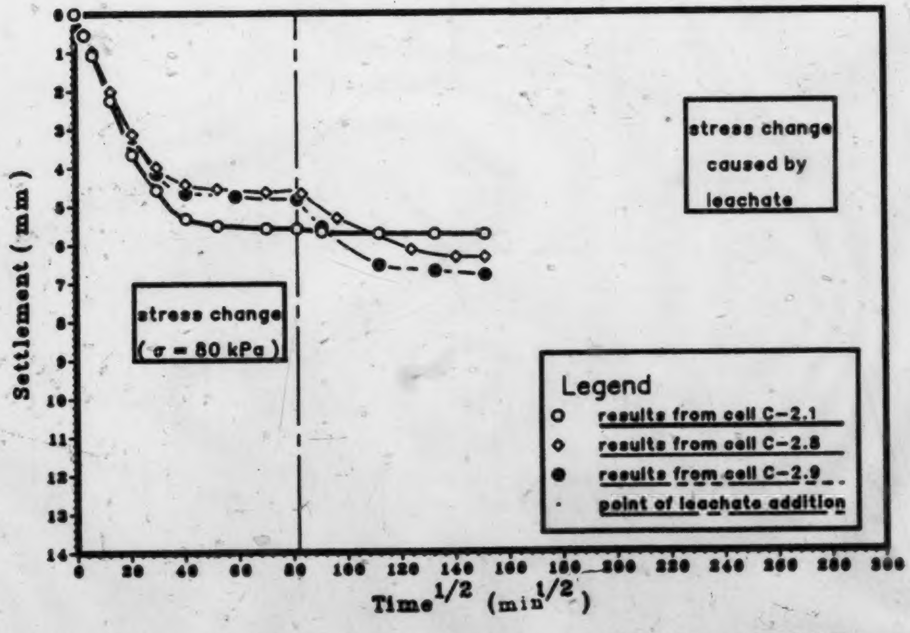

Figure E-2.1a Effect of Leachate Addition at 80 kPa

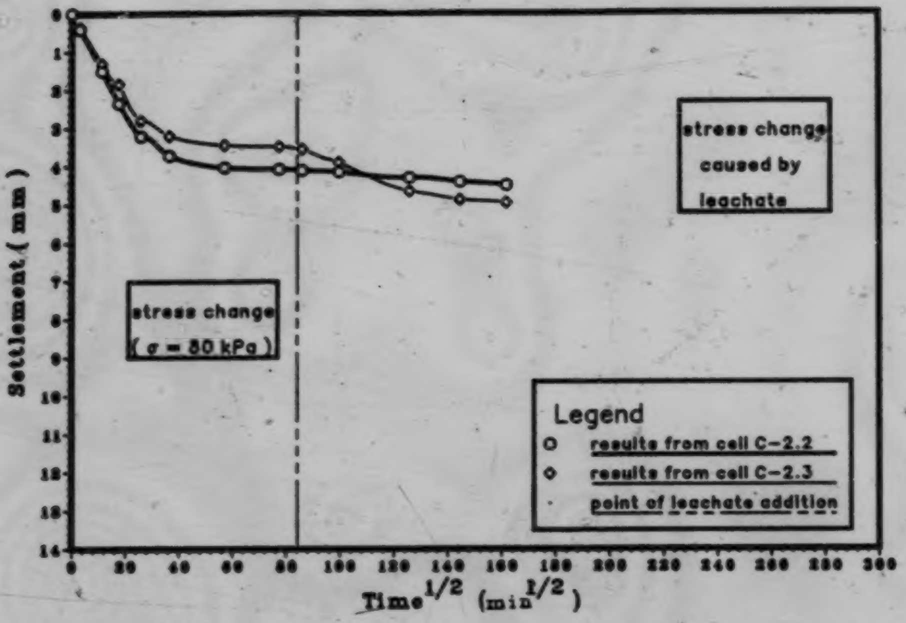

Figure E-2.2a Effect of Leachate Addition at 80 kPa

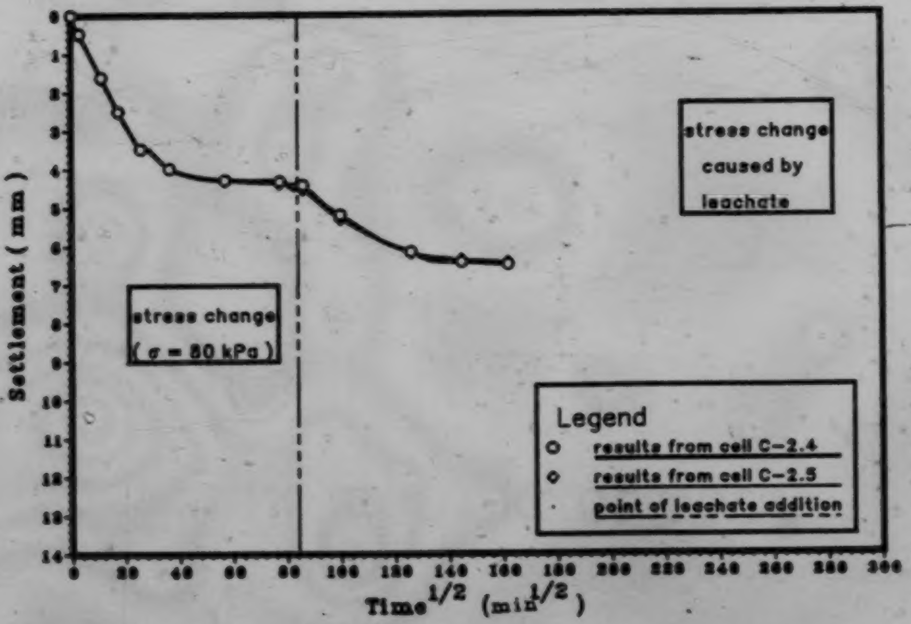

Figure E-2.3a Effect of Leachate Addition at 80 kPa

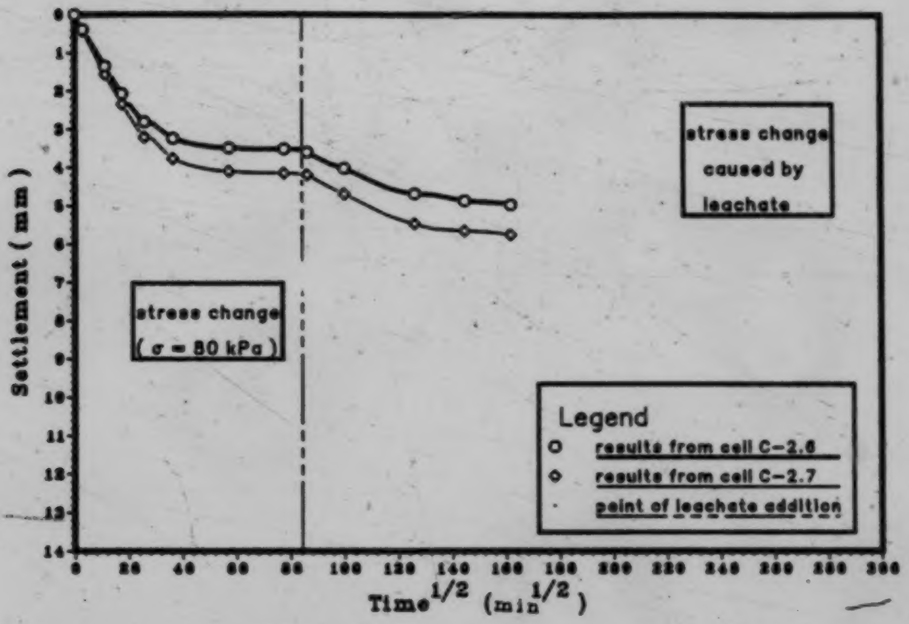

Figure E-2.4a Effect of Leachate Addition at 80 kPa

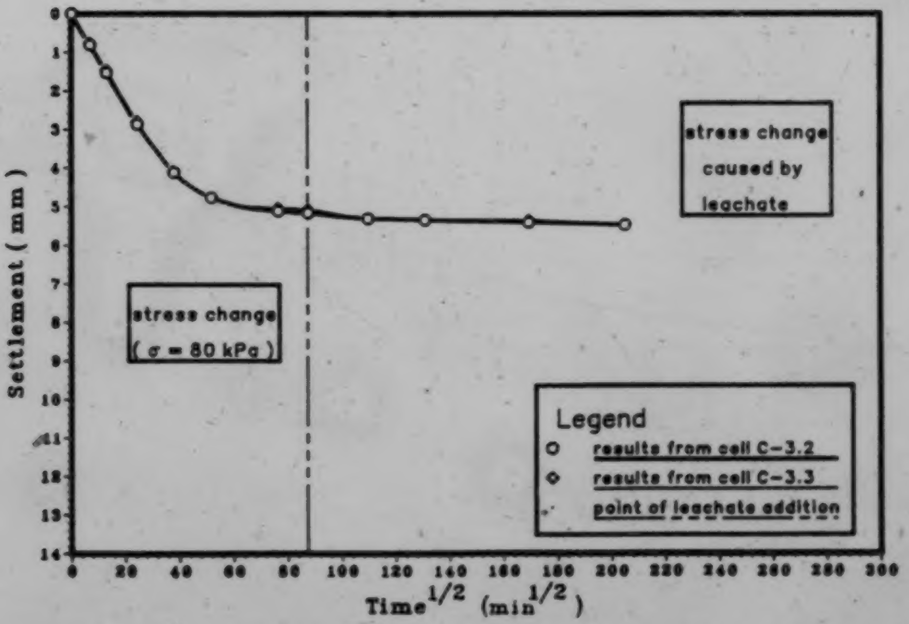

Figure E-3.1a Effect of Leachate Addition at 80 kPa

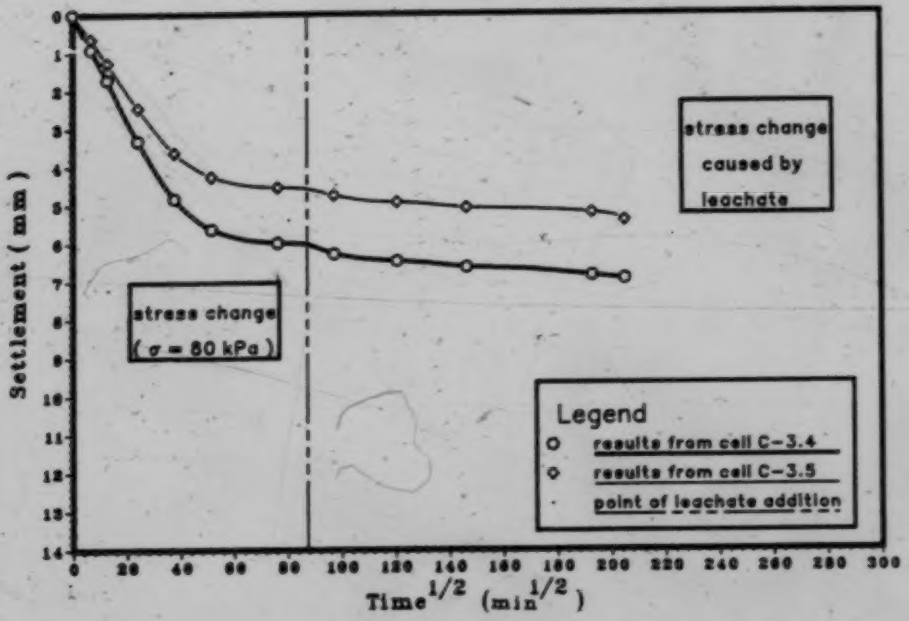

Figure E-3.2a Effect of Leachate Addition at 80 kPa

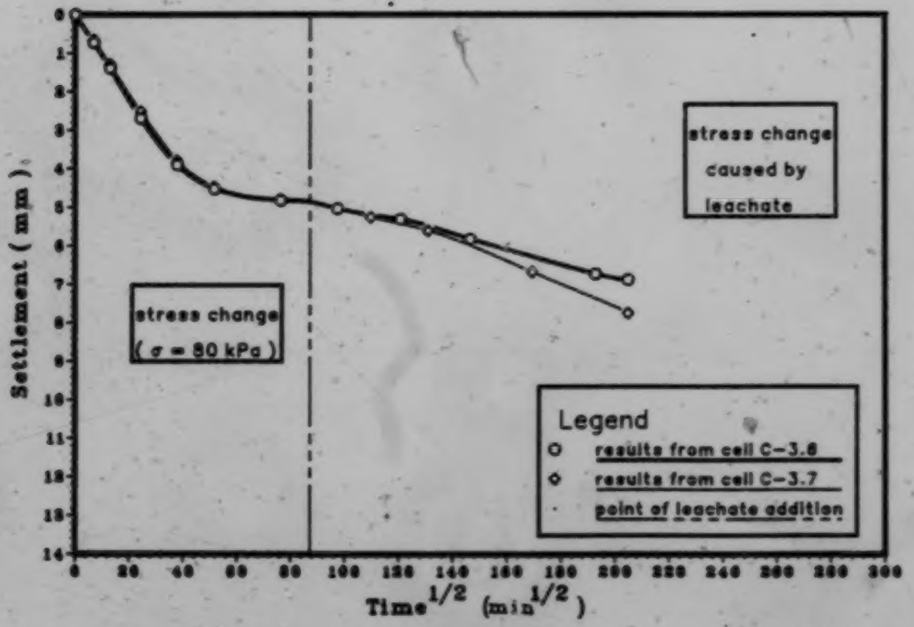

Figure E-3.3a Effect of Leachate Addition at 80 kPa

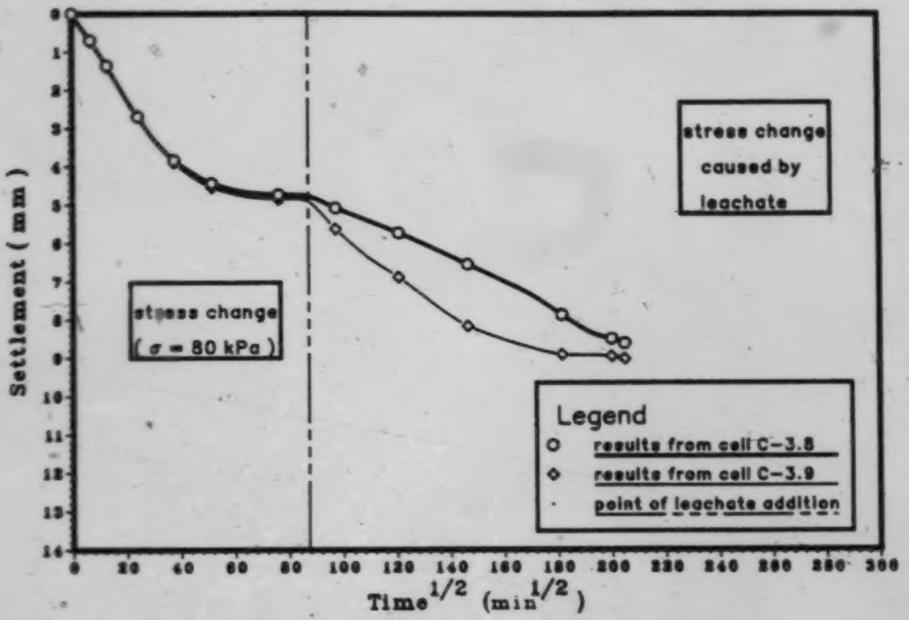

Figure E-3.4a Effect of Leachate Addition at 80 kPa

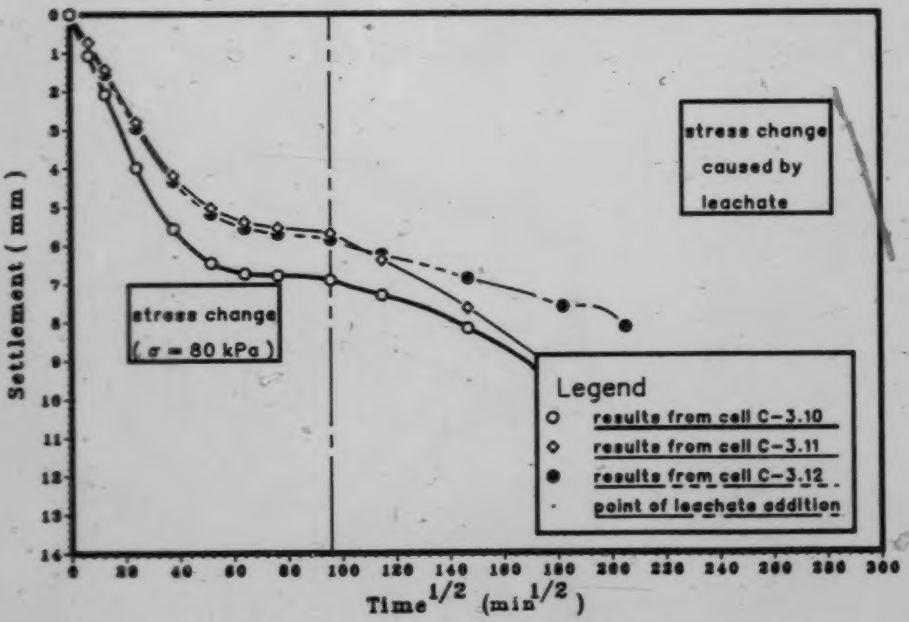

Figure E-3.5a Effect of Leachate Addition at 80 kPa

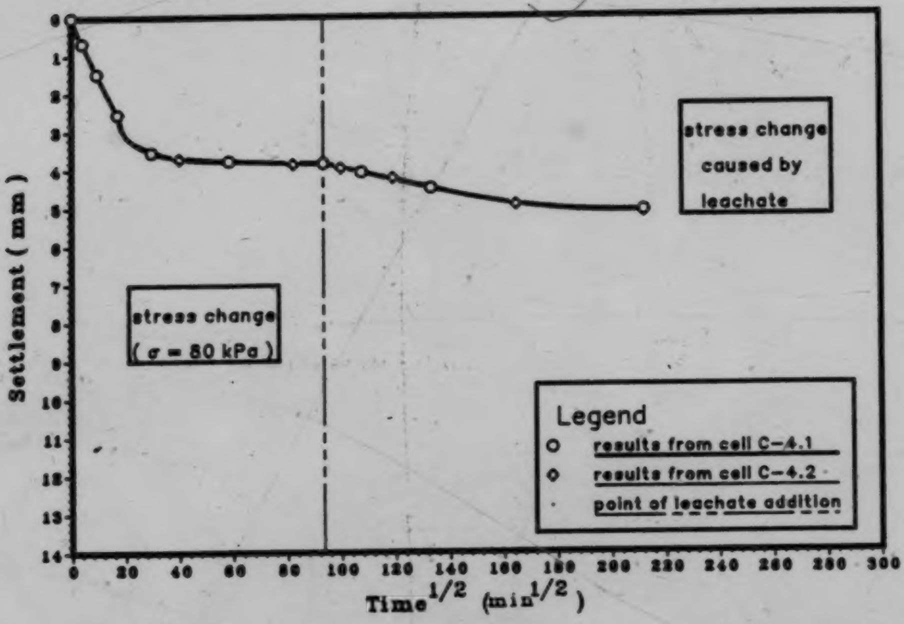

Figure E-4.1a Effect of Leachate Addition at 80 kPa

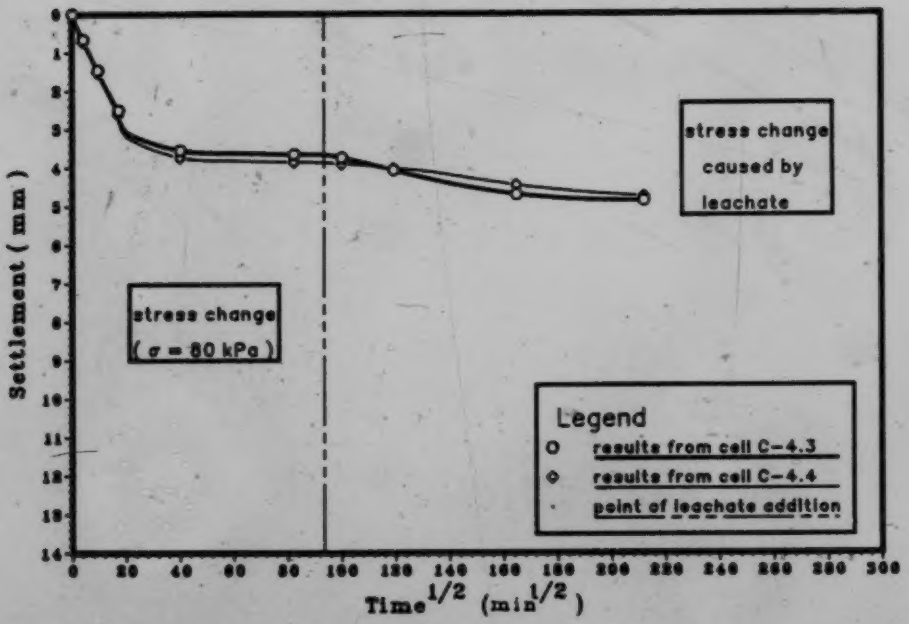

Figure E-4.2a Effect of Leachate Addition at 80 kPa

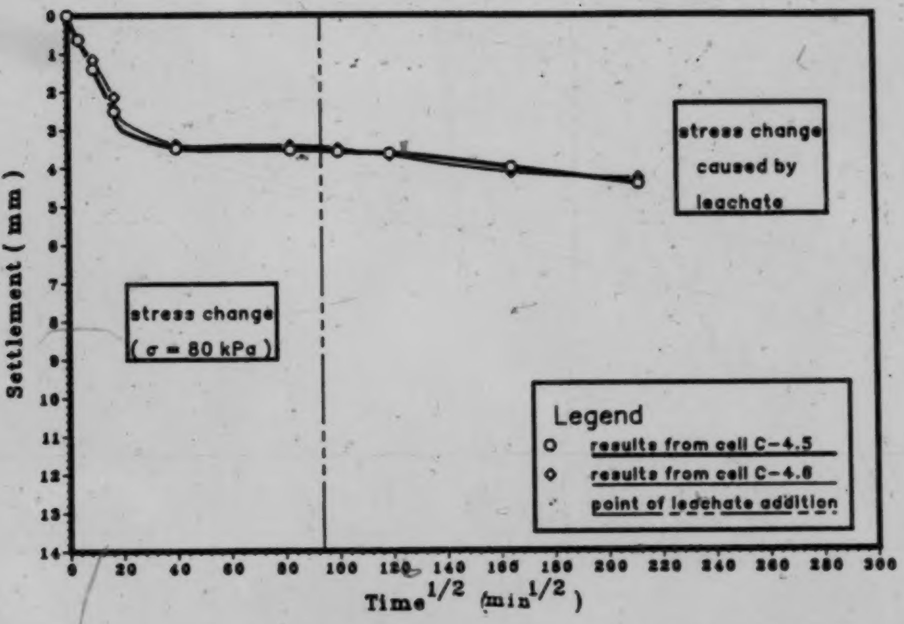

Figure E-4.3a Effect of Leachate Addition at 80 kPa

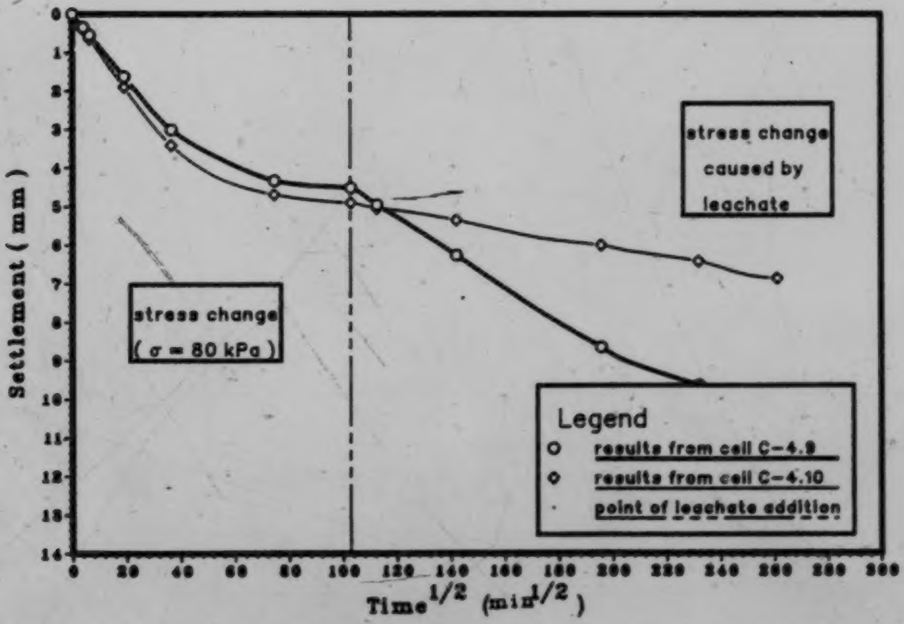

Figure E-4.5a Effect of Leachate Addition at 80 kPa

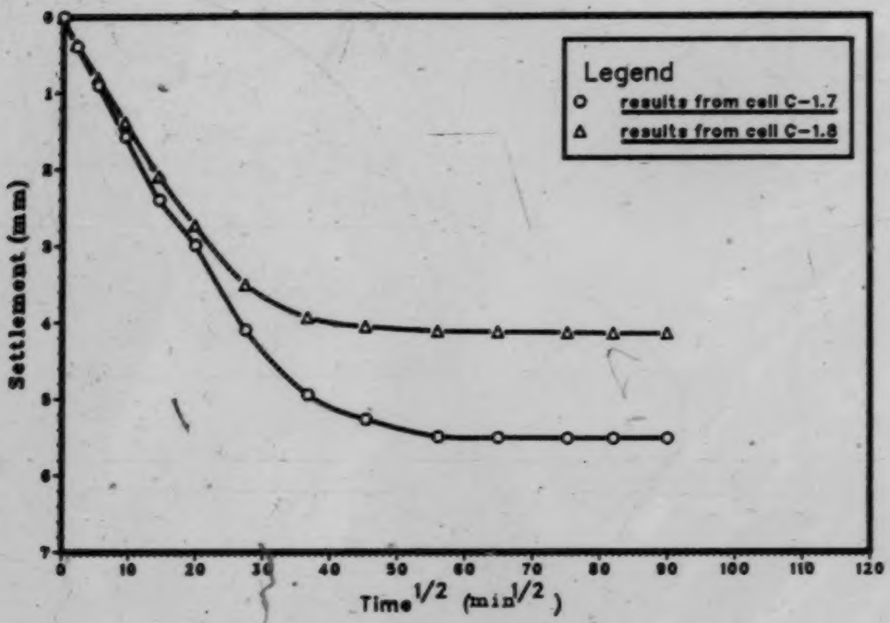

Figure E-1.3 Time Settlement Curve at 160 kPa

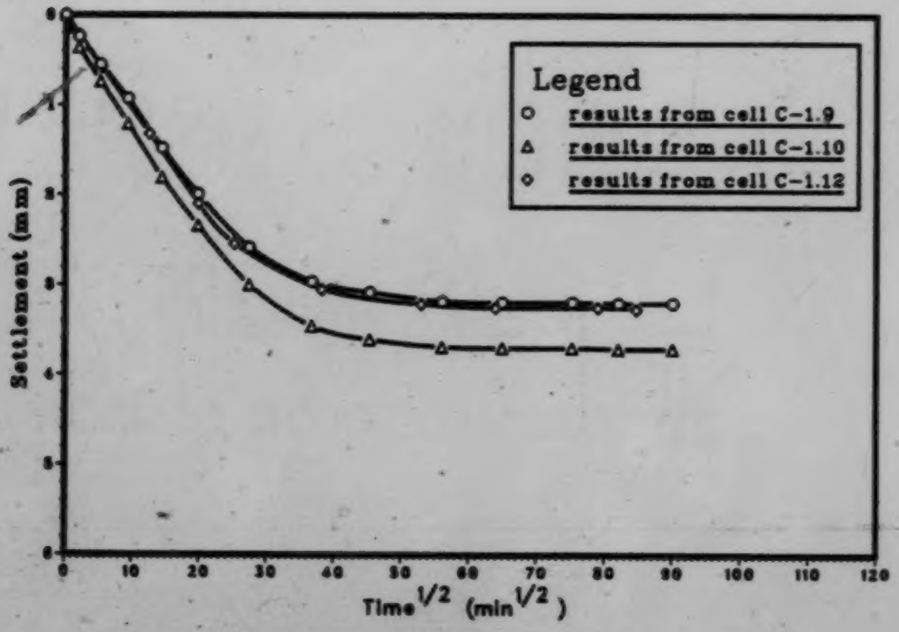

Figure E-1.4b Time Settlement Curve at 160 kPa

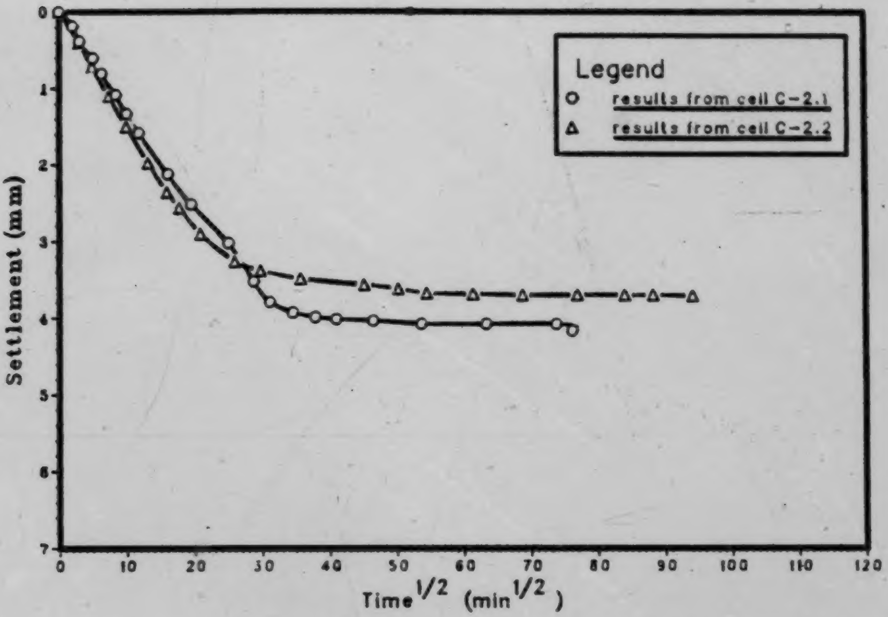

Figure E-2.1b Time Settlement Curve at 160 kPa

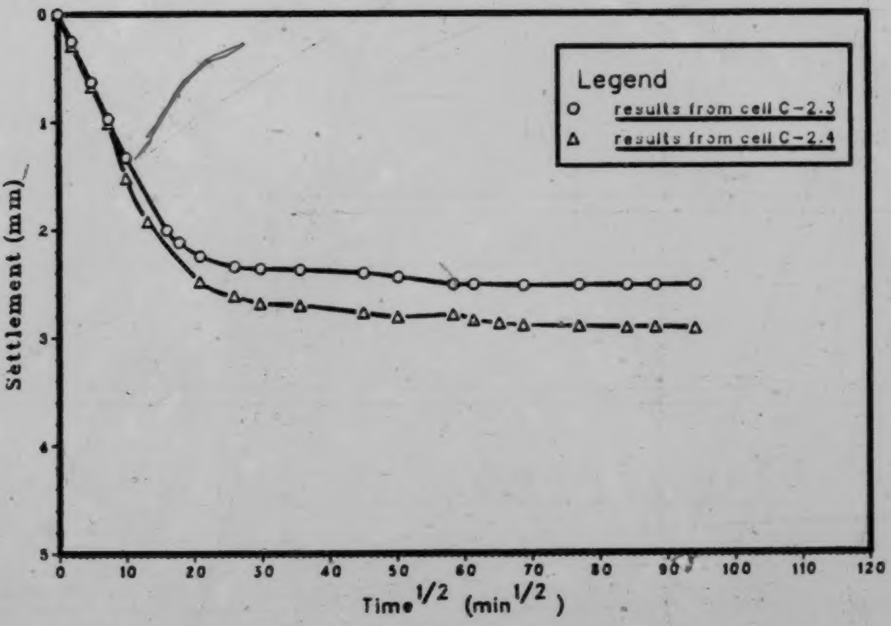

Figure E-2.2b Time Settlement Curve at 160 kPa

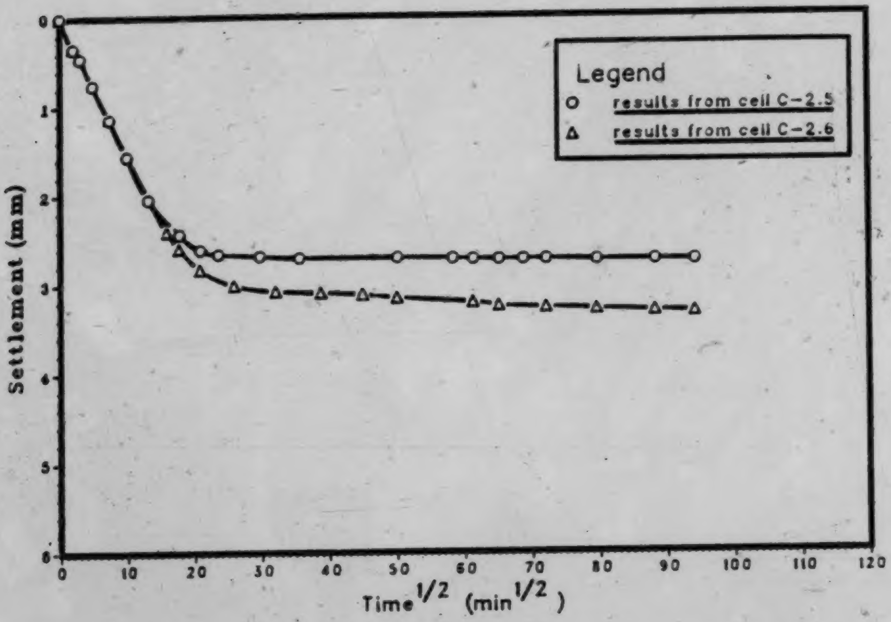

Figure E-2.3b Time Settlement Curve at 160 kPa

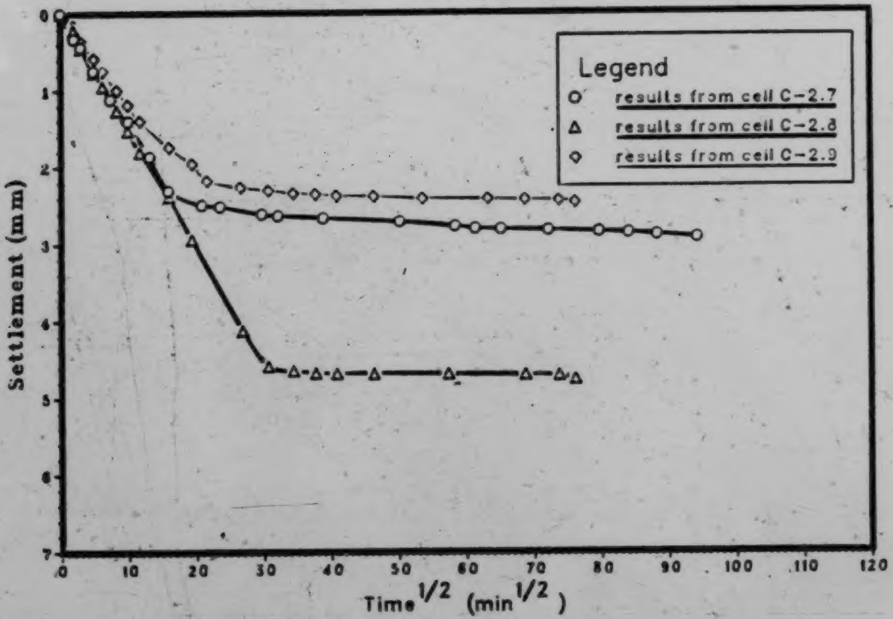

Figure E-2.4b Time Settlement Curve at 160 kPa

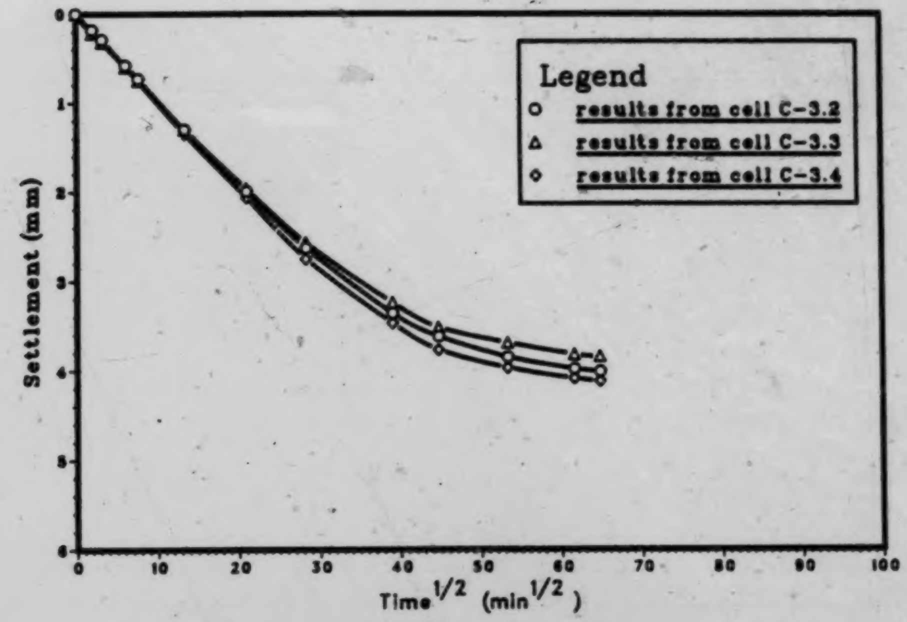

Figure E-3.2b Time Settlement Curves at 160 kPa

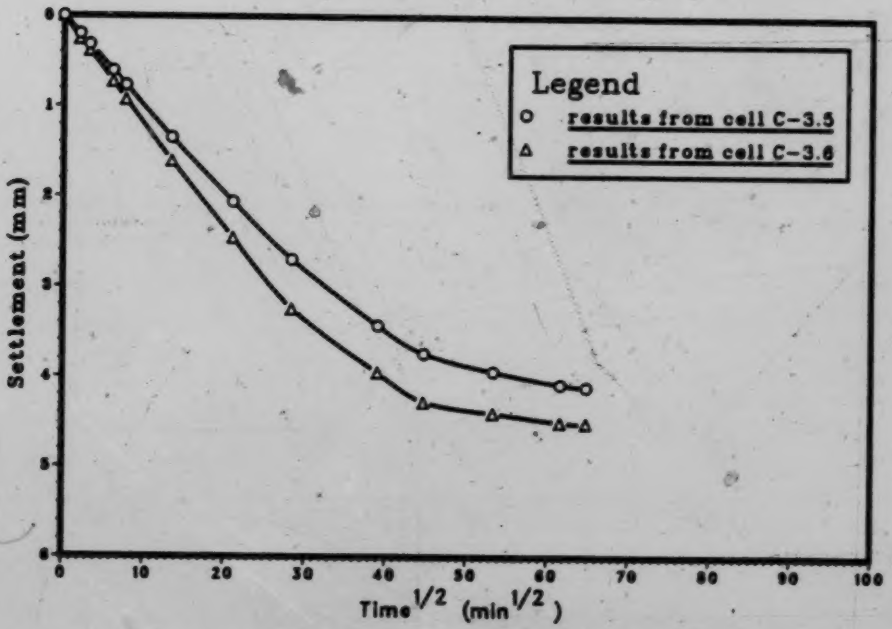

Figure E-3.3b Time Settlement Curves at 160 kPa

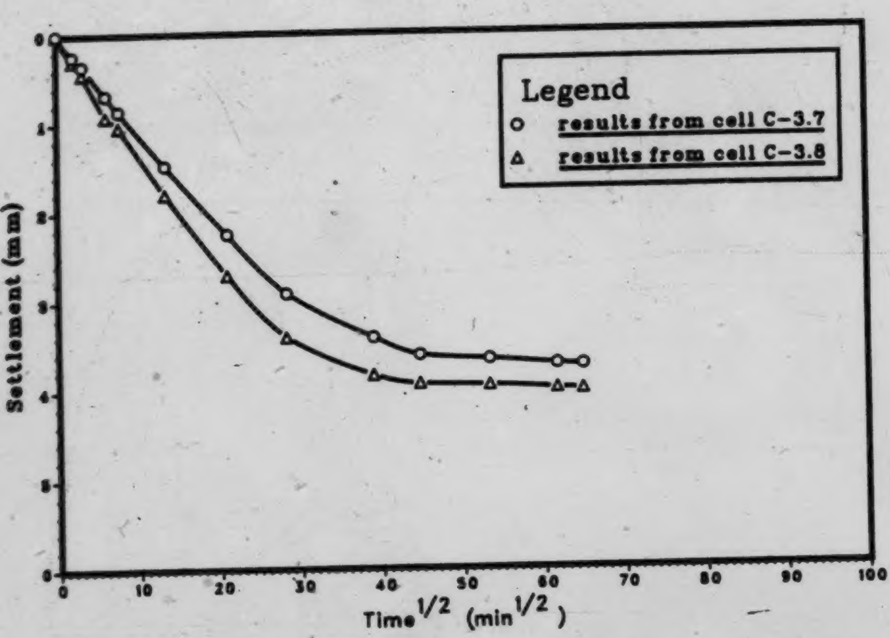

Figure E-3.4b Time Settlement Curves at 160 kPa

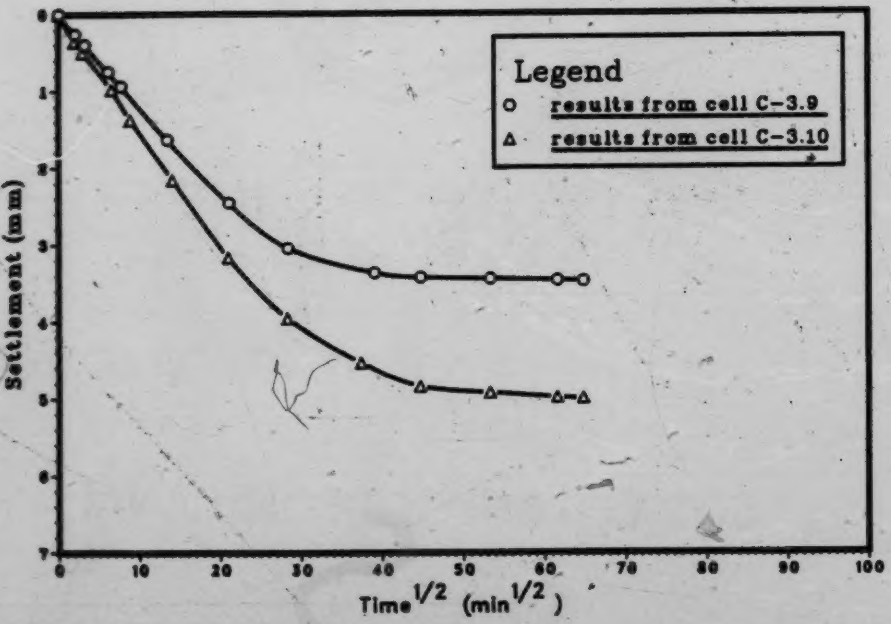

Figure E-3.5b Time Settlement Curves at 160 kPa

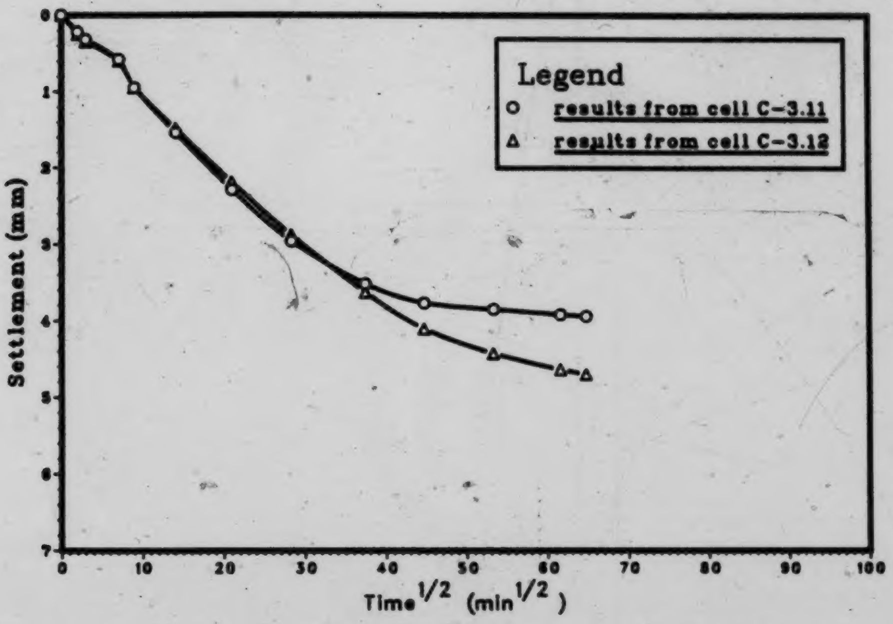

Figure E-3.6b Time Settlement Curve at 160 kPa

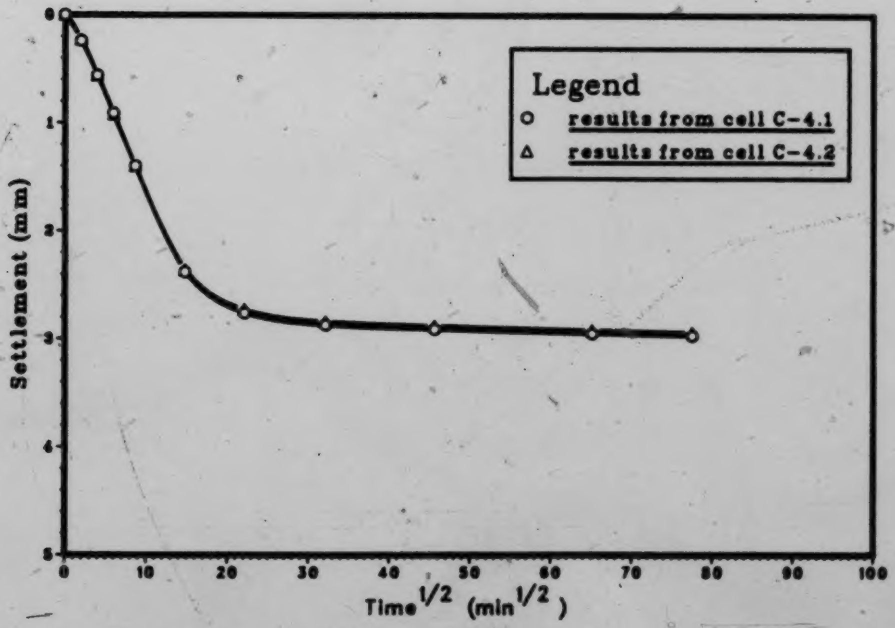

Figure E-4.1b Time Settlement Curve at 160 kPa

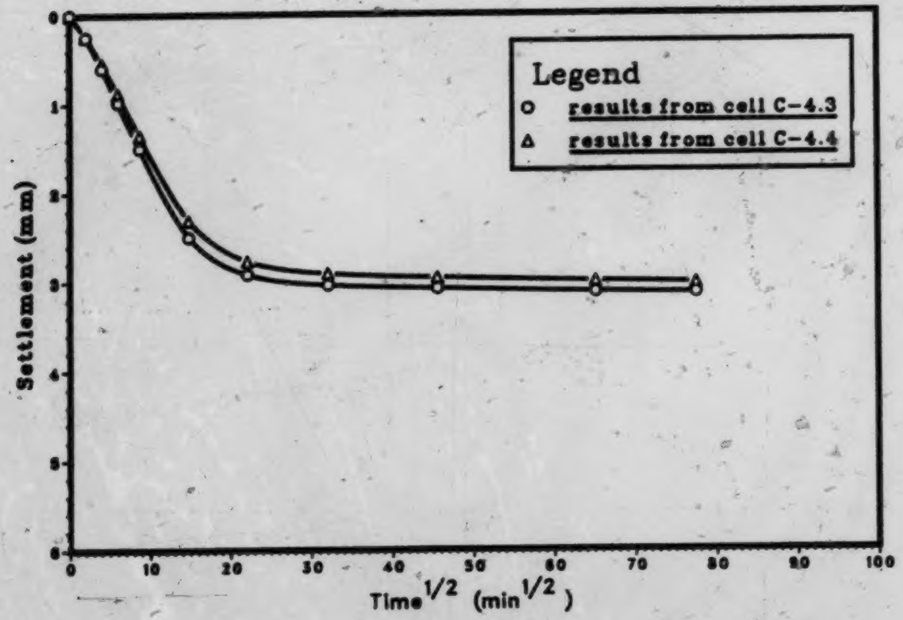

Figure E-4.2b Time Settlement Curve at 160 kPa

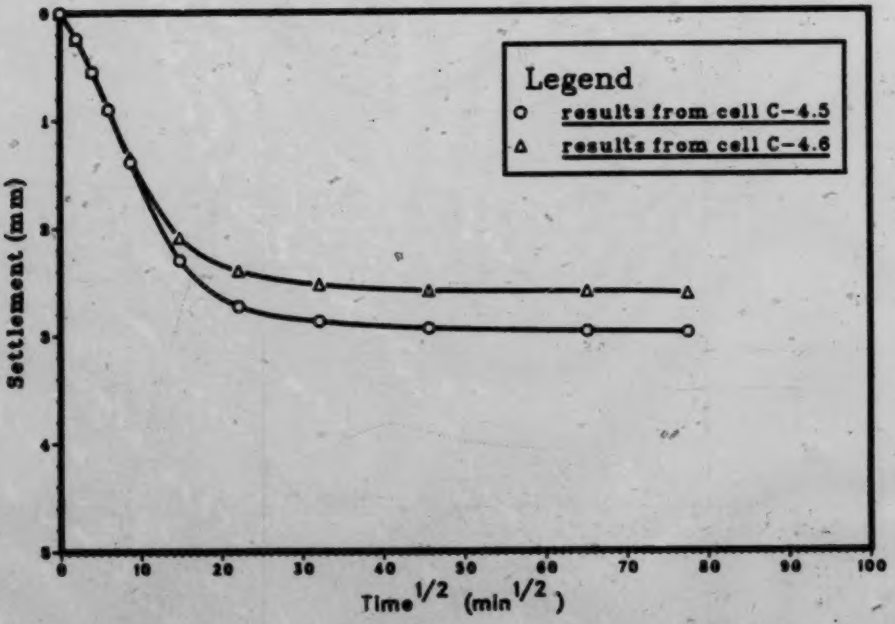

Figure E-4.3b Time Settlement Curve at 160 kPa

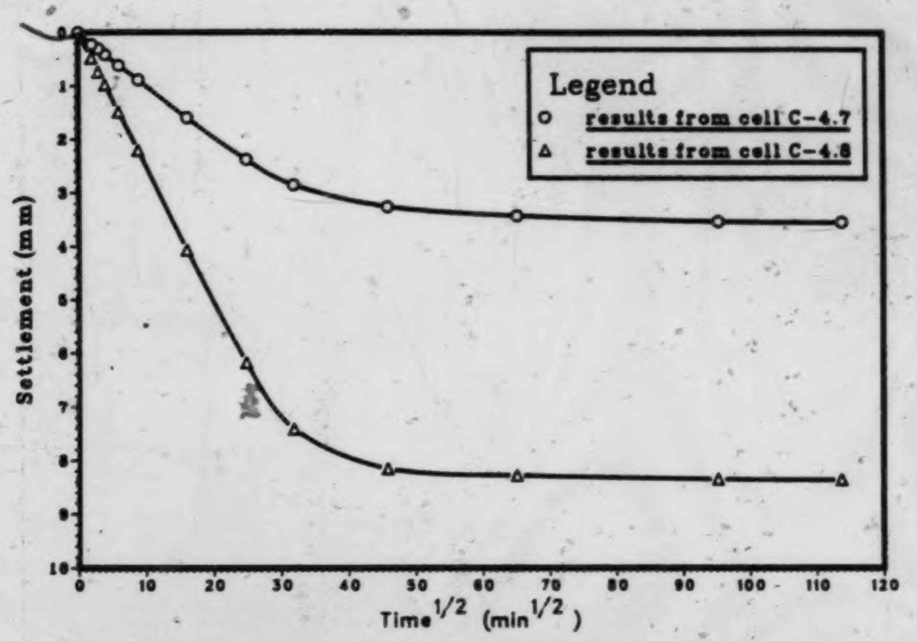

Figure E-4.4b Time Settlement Curve at 160 kPa

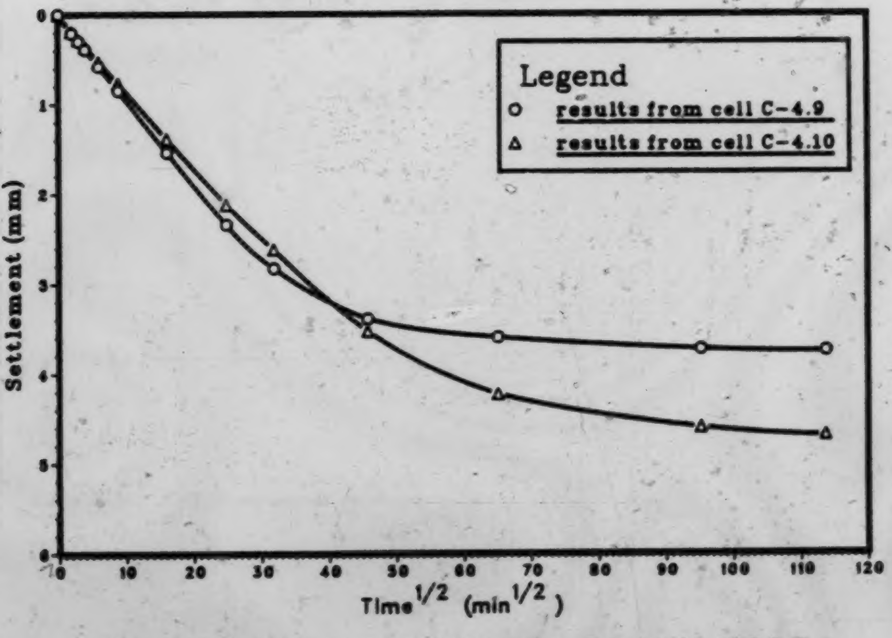

Figure E-4.5b Time Settlement Curve at 160 kPa

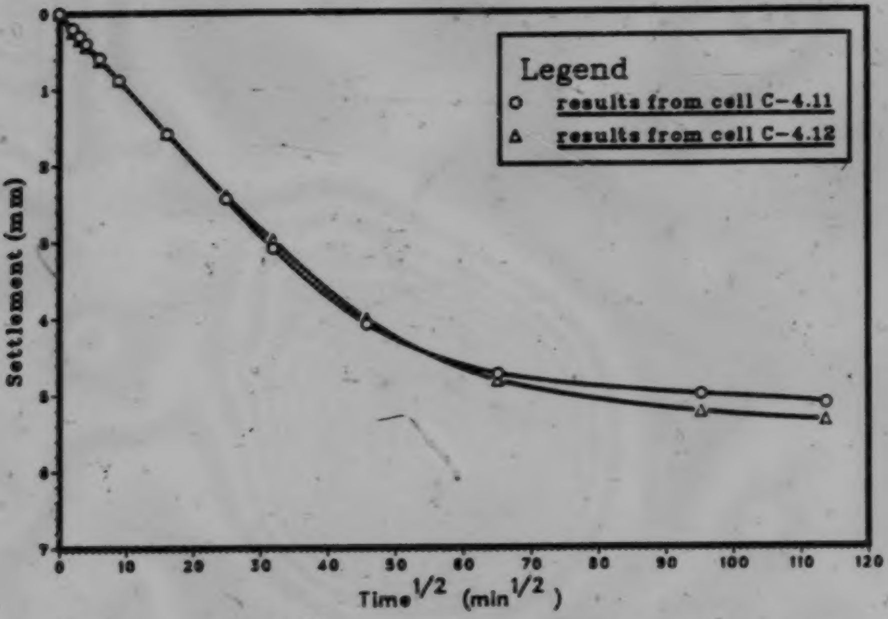

Figure E-4.6b Time Settlement Curve at 160 kPa

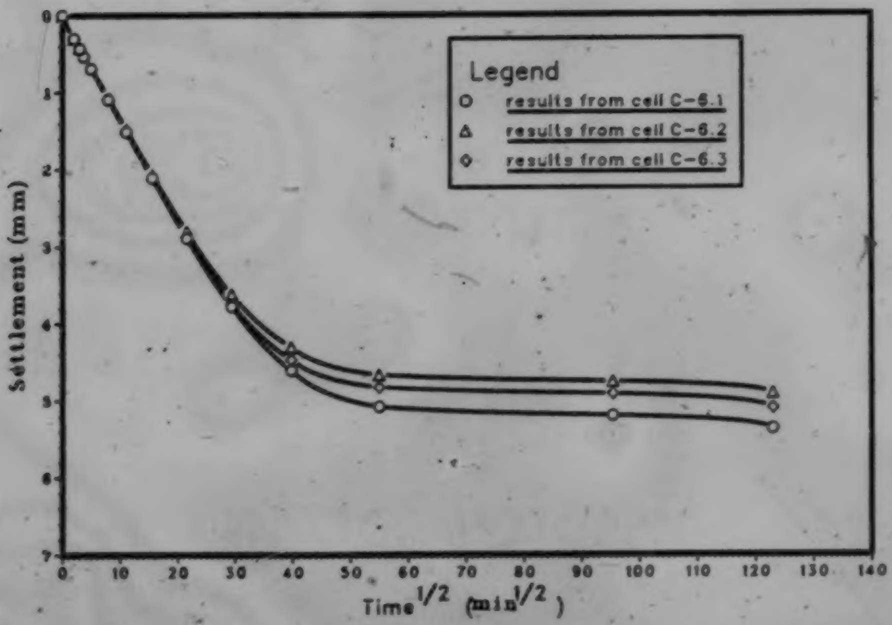

Figure E-6.1 Time Settlement Curve at 80 kPa (post-leachate)

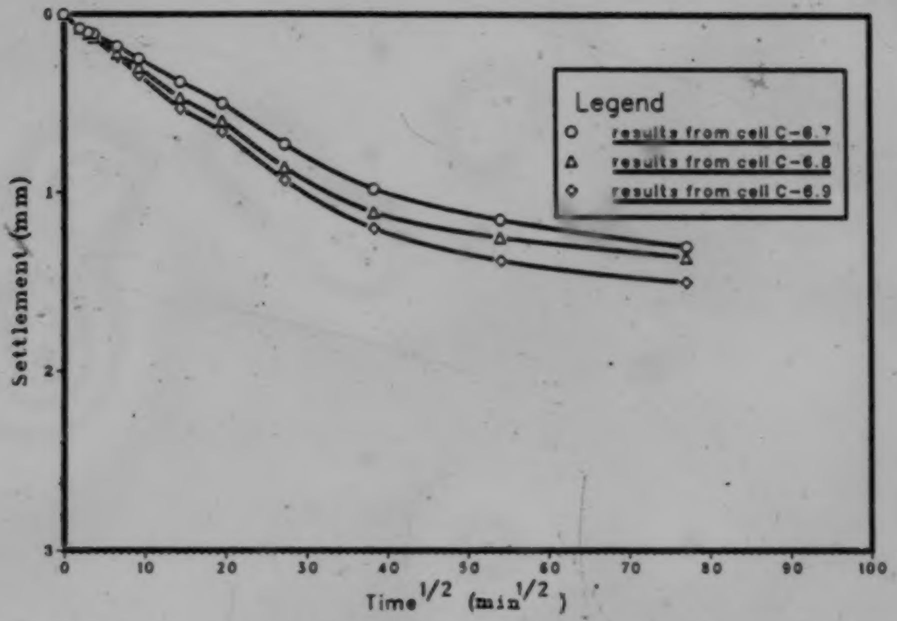

Figure E-6.2 Time Settlement Curve at 240 kPa (post-leachate)

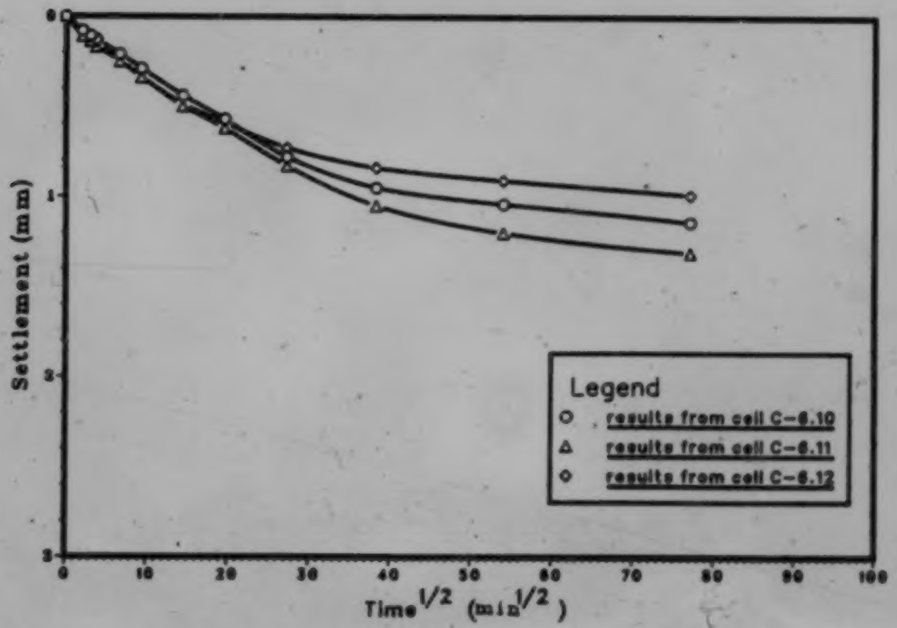

Figure E-6.3 Time Settlement Curve at 240 kPa (post-leachate)

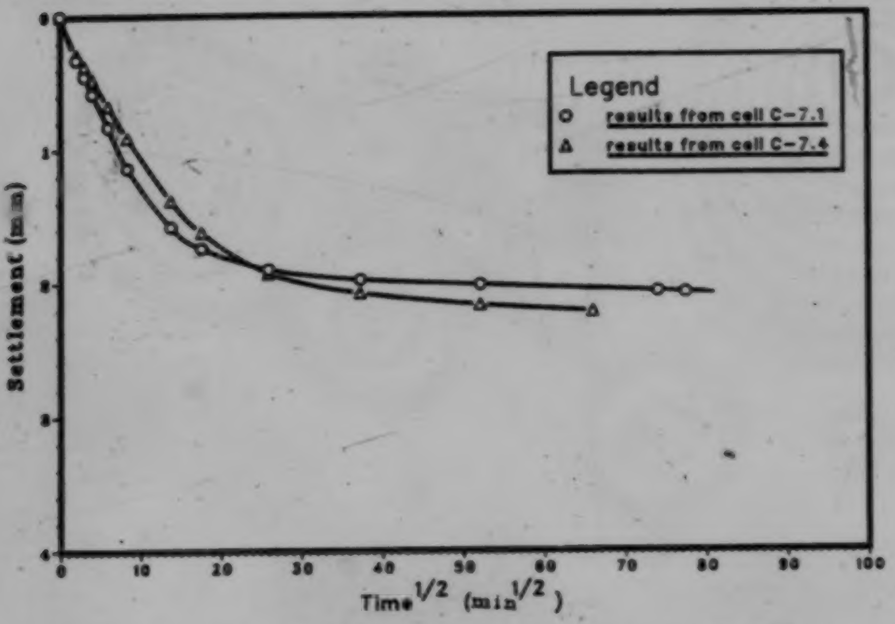

Figure E-7.1 Time Settlement Curve at 160 kPa

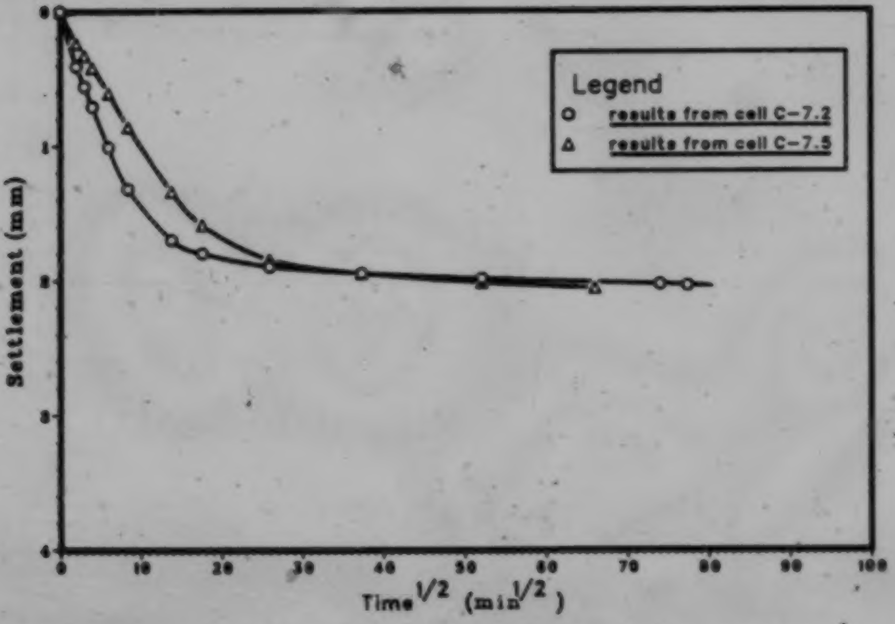

Figure E-7.2 Time Settlement Curve at 160 kPa

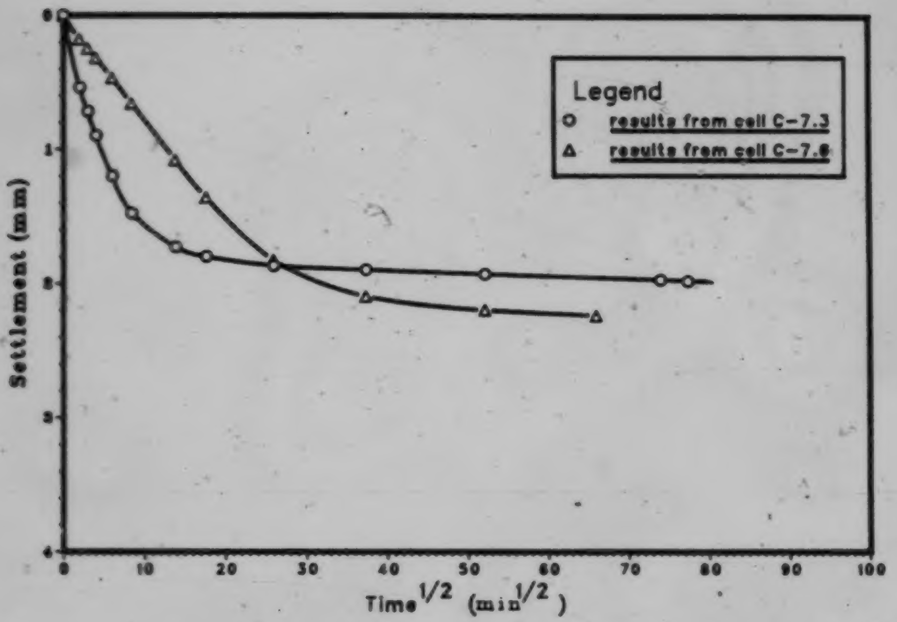

Figure E-7.3 Time Settlement Curve at 160 kPa

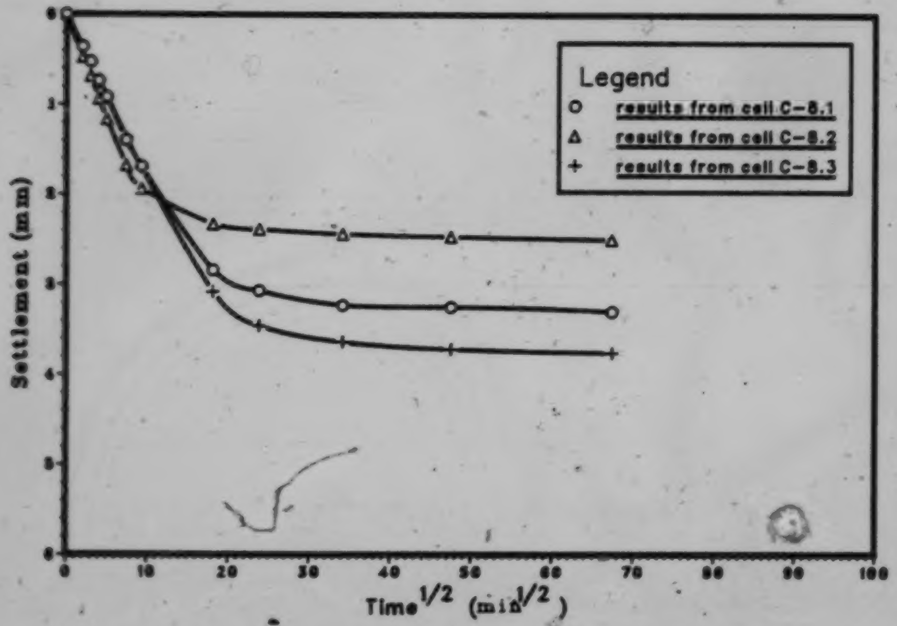

Figure E-8.1 Time Settlement Curve at 160 kPa

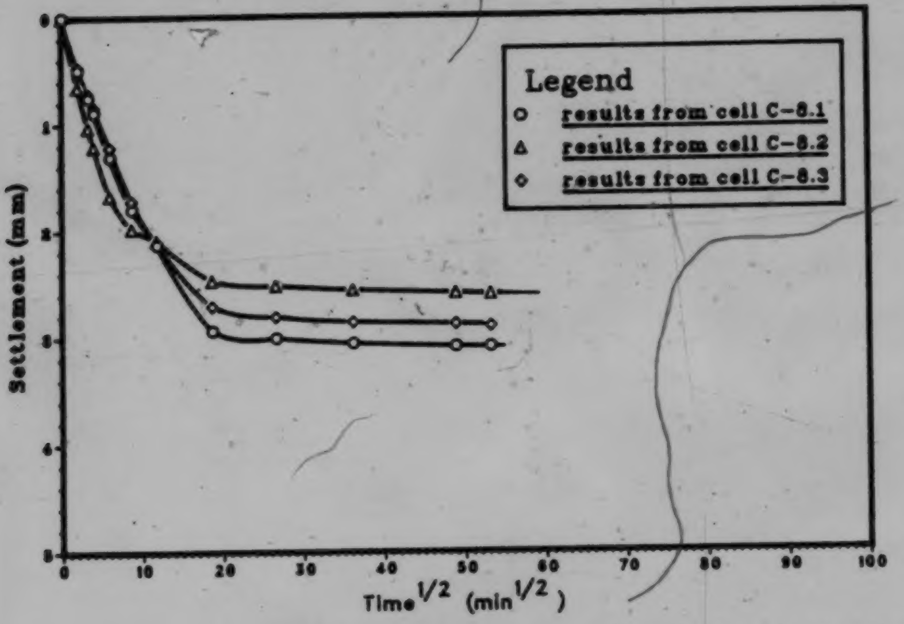

Figure E-8.2 Time Settlement Curve at 320 kPa (post-leachate)

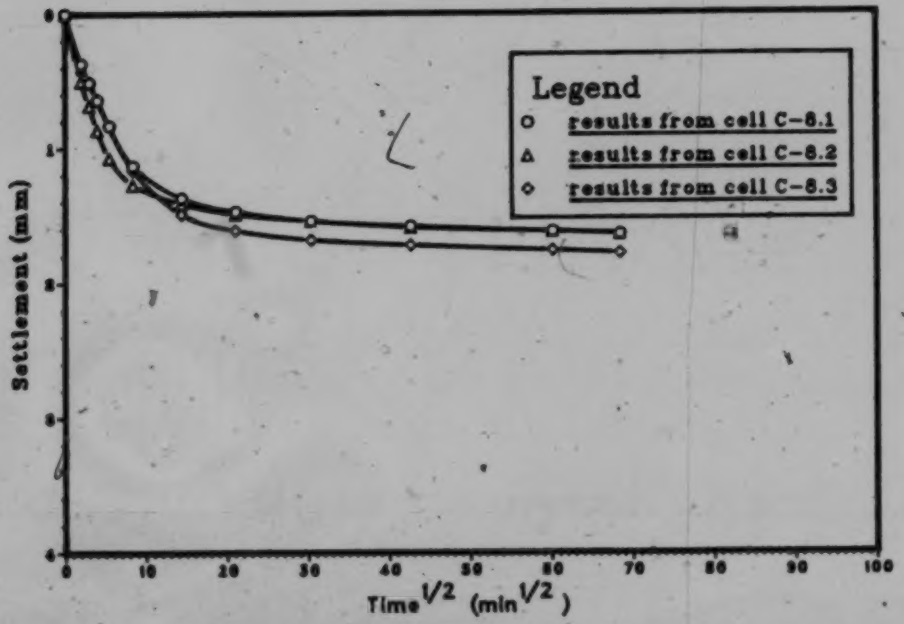

Figure E-8.3 Time Settlement Curve at 560 kPa (post-leachate)

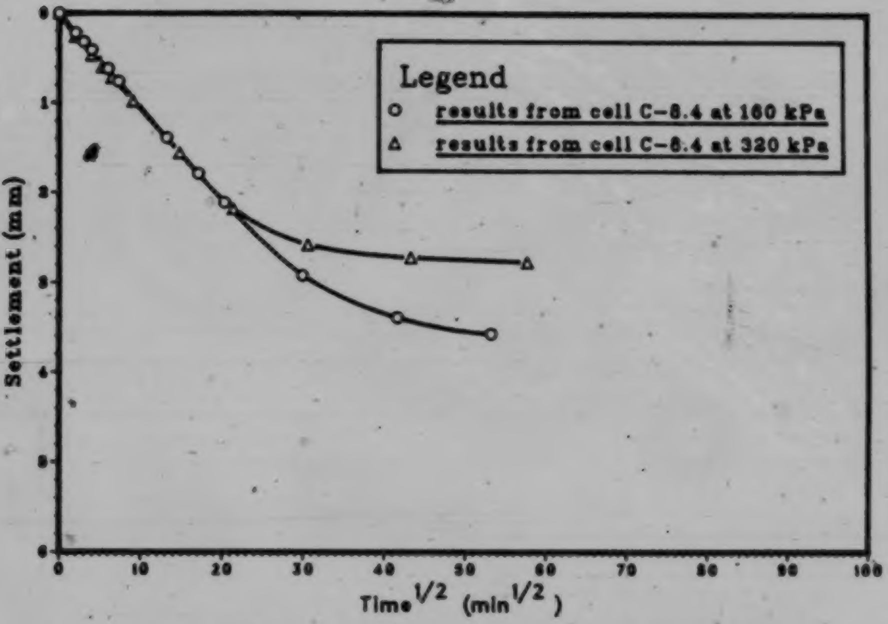

Figure E-8.4 Post-Leachate Time Settlement Curves

APPENDIX F - Void Ratio Vs log Pressure

TABLE F-1.1 VOID RATIO - PRESSURE RESULTS

(CELL C-1.2)

VOID RATIO (dimensionless)	PRESSURE (kPa)
13.93	1.0
10.66	10.0
9.42	20.0
8.05	40.0
6.56	80.0
6.10	Leachate
4.52	160.0 + Leachate

TABLE F-1.2 VOID RATIO - PRESSURE RESULTS

(CELL C-1.3)

VOID RATIO (dimensionless)	PRESSURE (kPa)
12.73	1.0
9.85	10.0
8.69	20.0
7.59	40.0
6.68	80.0
5.20	Leachate
3.88	160.0 + Leachate

TABLE F-1.3 VOID RATIO - PRESSURE RESULTS

(CELL C-1.5)

VOID RATIO (dimensionless)	PRESSURE (kPa)
13.03	1.0
10.28	10.0
9.16	20.0
6.75	40.0
5.70	80.0
4.24	Leachate
3.24	160.0 + Leachate

TABLE F-1.4 VOID RATIO - PRESSURE RESULTS

(CELL C-1.6)

VOID RATIO (dimensionless)	PRESSURE (kPa)
12.98	1.0
11.06	10.0
9.78	20.0
7.26	40.0
5.95	80.0
4.83	Leachate
3.62	160.0 + Leachate

TABLE F-1.5 VOID RATIO - PRESSURE RESULTS

(CELL C-1.7)

VOID RATIO (dimensionless)	PRESSURE (kPa)
14.38	1.0
12.17	10.0
10.70	20.0
7.33	40.0
6.07	80.0
5.02	Leachate
3.52	160.0 + Leachate

TABLE F-1.6 VOID RATIO - PRESSURE RESULTS

(CELL C-1.8)

VOID RATIO (dimensionless)	PRESSURE (kPa)
11.89	1.0
10.30	10.0
9.16	20.0
6.64	40.0
5.60	80.0
4.31	Leachate
3.20	160.0 + Leachate

TABLE F-1.7 VOID RATIO - PRESSURE RESULTS

(CELL C-1.9)

VOID RATIO (dimensionless)	PRESSURE (kPa)
12.20	1.0
9.77	10.0
9.04	20.0
6.71	40.0
5.56	80.0
4.12	Leachate
3.28	160.0 + Leachate

TABLE F-1.8 VOID RATIO - PRESSURE RESULTS

(CELL C-1.10)

VOID RATIO (dimensionless)	PRESSURE (kPa)
13.03	1.0
10.28	10.0
9.16	20.0
6.75	40.0
5.70	80.0
4.24	Leachate
3.24	160.0 + Leachate

TABLE F-1.9 VOID RATIO - PRESSURE RESULTS

(CELL C-1.11)

VOID RATIO (dimensionless)	PRESSURE (kPa)
12.85	1.0
10.13	10.0
8.93	20.0
6.76	40.0
5.78	80.0
4.29	Leachate
3.29	160.0 + Leachate

TABLE F-2.1 VOID RATIO - PRESSURE RESULTS

(CELL C-2.1)

VOID RATIO (dimensionless)	PRESSURE (kPa)
4.72	1.0
4.20	10.0
3.82	20.0
3.09	40.0
2.50	80.0
2.49	de-ionized water
2.03	160.0

TABLE F-2.2 VOID RATIO - PRESSURE RESULTS

(CELL C-2.2)

VOID RATIO (dimensionless)	PRESSURE (kPa)
5.47	1.0
4.34	10.0
4.06	20.0
3.69	40.0
3.24	80.0
3.19	Leachate
2.77	160.0 + Leachate

TABLE F-2.3 VOID RATIO - PRESSURE RESULTS

(CELL C-2.3)

VOID RATIO (dimensionless)	PRESSURE (kPa)
4.25	1.0
3.17	10.0
2.83	20.0
2.52	40.0
2.13	80.0
1.97	Leachate
1.68	160.0 + Leachate

TABLE F-2.4 VOID RATIO - PRESSURE RESULTS

(CELL C-2.4)

VOID RATIO (dimensionless)	PRESSURE (kPa)
4.37	1.0
3.58	10.0
3.09	20.0
2.68	40.0
2.20	80.0
1.96	Leachate
1.63	160.0 + Leachate

TABLE F-2.5 VOID RATIO - PRESSURE RESULTS

(CELL C-2.5)

VOID RATIO (dimensionless)	PRESSURE (kPa)
4.70	1.0
3.74	10.0
3.28	20.0
2.92	40.0
2.43	80.0
2.20	Leachate
1.90	160.0 + Leachate

TABLE F-2.6 VOID RATIO - PRESSURE RESULTS

(CELL C-2.6)

VOID RATIO (dimensionless)	PRESSURE (kPa)
4.67	1.0
3.56	10.0
3.05	20.0
2.71	40.0
2.32	80.0
2.15	Leachate
1.78	160.0 + Leachate

TABLE F-2.7 VOID RATIO - PRESSURE RESULTS

(CELL C-2.7)

VOID RATIO (dimensionless)	PRESSURE (kPa)
4.40	1.0
3.76	10.0
3.21	20.0
2.76	40.0
2.30	80.0
2.12	Leachate
1.79	160.0 + Leachate

TABLE F-2.8 VOID RATIO - PRESSURE RESULTS

(CELL C-2.8)

VOID RATIO (dimensionless)	PRESSURE (kPa)
4.50	1.0
3.89	10.0
3.55	20.0
3.04	40.0
2.53	80.0
2.33	Leachate
1.80	160.0 + Leachate

TABLE F-2.9 VOID RATIO - PRESSURE RESULTS

(CELL C-2.9)

VOID RATIO (dimensionless)	PRESSURE (kPa)
4.41	1.0
3.93	10.0
3.35	20.0
2.78	40.0
2.26	80.0
2.04	Leachate
1.76	160.0 + Leachate

TABLE F-3.1 VOID RATIO - PRESSURE RESULTS

(CELL C-3.1)

VOID RATIO (dimensionless)	PRESSURE (kPa)
8.32	1.0
7.07	10.0
5.41	20.0
4.22	40.0
4.06	80.0
4.03	de-ionized water

TABLE F-3.2 VOID RATIO - PRESSURE RESULTS

(CELL C-3.2)

VOID RATIO (dimensionless)	PRESSURE (kPa)
6.10	1.0
4.83	10.0
4.27	20.0
3.68	40.0
2.94	80.0
2.91	Leachate
2.36	160.0 + Leachate

TABLE F-3.3 VOID RATIO - PRESSURE RESULTS

(CELL C-3.3)

VOID RATIO (dimensionless)	PRESSURE (kPa)
5.88	1.0
4.77	10.0
4.15	20.0
3.53	40.0
2.80	80.0
2.75	Leachate
2.20	160.0 + Leachate

TABLE F-3.4 VOID RATIO - PRESSURE RESULTS

(CELL C-3.4)

VOID RATIO (dimensionless)	PRESSURE (kPa)
6.28	1.0
5.07	10.0
4.45	20.0
3.65	40.0
2.78	80.0
2.73	Leachate
2.14	160.0 + Leachate

TABLE F-3.5 VOID RATIO - PRESSURE RESULTS

(CELL C-3.5)

VOID RATIO (dimensionless)	PRESSURE (kPa)
6.19	1.0
4.60	10.0
4.16	20.0
3.51	40.0
2.85	80.0
2.73	Leachate
2.14	160.0 + Leachate

TABLE F-3.6 VOID RATIO - PRESSURE RESULTS

(CELL C-3.6)

VOID RATIO (dimensionless)	PRESSURE (kPa)
6.26	1.0
4.80	10.0
4.24	20.0
3.56	40.0
2.86	80.0
2.57	Leachate
1.92	160.0 + Leachate

TABLE F-3.7 VOID RATIO - PRESSURE RESULTS

(CELL C-3.7)

VOID RATIO (dimensionless)	PRESSURE (kPa)
6.28	1.0
4.73	10.0
4.13	20.0
3.49	40.0
2.79	80.0
2.37	Leachate
1.83	160.0 + Leachate

TABLE F-3.8 VOID RATIO - PRESSURE RESULTS

(CELL C-3.8)

VOID RATIO (dimensionless)	PRESSURE (kPa)
6.49	1.0
5.05	10.0
4.25	20.0
3.53	40.0
2.84	80.0
2.32	Leachate
1.74	160.0 + Leachate

TABLE F-3.9 VOID RATIO - PRESSURE RESULTS

(CELL C-3.9)

VOID RATIO (dimensionless)	PRESSURE (kPa)
6.32	1.0
4.68	10.0
4.21	20.0
3.53	40.0
2.83	80.0
2.24	Leachate
1.74	160.0 + Leachate

TABLE F-3.10 VOID RATIO - PRESSURE RESULTS

(CELL C-3.10)

VOID RATIO (dimensionless)	PRESSURE (kPa)
9.99	1.0
8.09	10.0
7.42	20.0
5.77	40.0
4.22	80.0
3.14	Leachate
2.02	160.0 + Leachate

TABLE F-3.11 VOID RATIO - PRESSURE RESULTS

(CELL C-3.11)

VOID RATIO (dimensionless)	PRESSURE (kPa)
9.48	1.0
7.49	10.0
6.43	20.0
5.20	40.0
3.92	80.0
2.93	Leachate
2.05	160.0 + Leachate

TABLE F-3.12 VOID RATIO - PRESSURE RESULTS

(CELL C-3.12)

VOID RATIO (dimensionless)	PRESSURE (kPa)
9.52	1.0
7.65	10.0
6.67	20.0
5.37	40.0
4.06	80.0
3.56	Leachate
2.51	160.0 + Leachate

V

TABLE F-4.1 VOID RATIO - PRESSURE RESULTS

(CELL C-4.1)

VOID RATIO (dimensionless)	PRESSURE (kPa)
3.98	1.0
3.17	10.0
2.78	20.0
2.41	40.0
2.04	80.0
1.92	Leachate
1.63	160.0 + Leachate

TABLE F-4.2 VOID RATIO - PRESSURE RESULTS

(CELL C-4.2)

VOID RATIO (dimensionless)	PRESSURE (kPa)
3.95	1.0
3.16	10.0
2.78	20.0
2.41	40.0
2.04	80.0
1.92	Leachate
1.64	160.0 + Leachate

TABLE F-4.3 VOID RATIO - PRESSURE RESULTS

(CELL C-4.3)

VOID RATIO (dimensionless)	PRESSURE (kPa)
3.96	1.0
3.31	10.0
2.73	20.0
2.41	40.0
2.06	80.0
1.95	Leachate
1.65	160.0 + Leachate

TABLE F-4.4 VOID RATIO - PRESSURE RESULTS

(CELL C-4.4)

VOID RATIO (dimensionless)	PRESSURE (kPa)
3.98	1.0
3.05	10.0
2.66	20.0
2.32	40.0
1.95	80.0
1.87	Leachate
1.58	160.0 + Leachate

TABLE F-4.5 VOID RATIO - PRESSURE RESULTS

(CELL C-4.5)

VOID RATIO (dimensionless)	PRESSURE (kPa)
3.61	1.0
2.94	10.0
2.67	20.0
2.34	40.0
2.00	80.0
1.92	Leachate
1.64	160.0 + Leachate

TABLE F-4.6 VOID RATIO - PRESSURE RESULTS

(CELL C-4.6)

VOID RATIO (dimensionless)	PRESSURE (kPa)
3.35	1.0
3.04	10.0
2.82	20.0
2.35	40.0
2.02	80.0
1.94	Leachate
1.69	160.0 + Leachate

TABLE F-4.7 VOID RATIO - PRESSURE RESULTS

(CELL C-4.7)

VOID RATIO (dimensionless)	PRESSURE (kPa)
13.04	1.0
9.01	10.0
8.10	20.0
6.75	40.0
5.30	80.0
3.68	Leachate
2.65	160.0 + Leachate

TABLE F-4.8 VOID RATIO - PRESSURE RESULTS

(CELL C-4.8)

VOID RATIO (dimensionless)	PRESSURE (kPa)
13.92	1.0
11.40	10.0
8.71	20.0
6.75	40.0
5.57	80.0
5.15	Leachate
2.72	160.0 + Leachate

TABLE F-4.9 VOID RATIO - PRESSURE RESULTS

(CELL C-4.9)

VOID RATIO (dimensionless)	PRESSURE (kPa)
12.86	1.0
8.90	10.0
8.10	20.0
6.79	40.0
5.48	80.0
3.87	Leachate
2.78	160.0 + Leachate

TABLE F-4.10 VOID RATIO - PRESSURE RESULTS

(CELL C-4.10)

VOID RATIO (dimensionless)	PRESSURE (kPa)
13.01	1.0
9.40	10.0
8.20	20.0
6.72	40.0
5.30	80.0
4.73	Leachate
3.37	160.0 + Leachate

TABLE F-4.11 VOID RATIO - PRESSURE RESULTS

(CELL C-4.11)

VOID RATIO (dimensionless)	PRESSURE (kPa)
14.33	1.0
13.51	10.0
13.37	20.0
8.39	40.0
5.80	80.0
4.39	Leachate
2.89	160.0 + Leachate

TABLE F-4.12 VOID RATIO - PRESSURE RESULTS

(CELL C-4.12)

VOID RATIO (dimensionless)	PRESSURE (kPa)
13.57	1.0
10.18	10.0
8.86	20.0
7.26	40.0
5.68	80.0
4.92	Leachate
3.40	160.0 + Leachate

TABLE F-5.1 VOID RATIO - PRESSURE RESULTS

(CELL C-5.1)

VOID RATIO (dimensionless)	PRESSURE (kPa)
15.46	1.0
13.80	10.0
10.36	20.0
8.40	40.0
6.59	80.0
4.40	Leachate

TABLE F-5.2 VOID RATIO - PRESSURE RESULTS

(CELL C-5.2)

VOID RATIO (dimensionless)	PRESSURE (kPa)
12.53	1.0
11.20	10.0
8.61	20.0
6.99	40.0
5.61	80.0
4.25	Leachate

TABLE F-5.3 VOID RATIO - PRESSURE RESULTS

(CELL C-5.3)

VOID RATIO (dimensionless)	PRESSURE (kPa)
10.58	1.0
9.25	10.0
6.98	20.0
5.73	40.0
4.73	80.0
3.88	Leachate

TABLE F-5.4 VOID RATIO - PRESSURE RESULTS

(CELL C-5.4)

VOID RATIO (dimensionless)	PRESSURE (kPa)
7.25	1.0
6.63	10.0
5.12	20.0
4.47	40.0
3.94	80.0
3.65	Leachate

TABLE F-5.5 VOID RATIO - PRESSURE RESULTS

(CELL C-5.5)

VOID RATIO (dimensionless)	PRESSURE (kPa)
5.28	1.0
4.67	10.0
3.83	20.0
3.42	40.0
3.06	80.0
2.97	Leachate

TABLE F-5.6 VOID RATIO - PRESSURE RESULTS

(CELL C-5.6)

VOID RATIO (dimensionless)	PRESSURE (kPa)
3.14	1.0
2.76	10.0
2.22	20.0
2.04	40.0
1.82	80.0
1.78	Leachate

TABLE F-5.7 VOID RATIO - PRESSURE RESULTS

(CELL C-5.7)

VOID RATIO (dimensionless)	PRESSURE (kPa)
2.69	1.0
2.46	10.0
1.99	20.0
1.83	40.0
1.63	80.0
1.61	Leachate

TABLE F-5.8 VOID RATIO - PRESSURE RESULTS

(CELL C-5.8)

VOID RATIO (dimensionless)	PRESSURE (kPa)
12.67	1.0
9.30	10.0
8.04	20.0
6.41	40.0
4.88	80.0
3.43	Leachate

TABLE F-5.9 VOID RATIO - PRESSURE RESULTS

(CELL C-5.9)

VOID RATIO (dimensionless)	PRESSURE (kPa)
2.28	1.0
1.96	10.0
1.38	20.0
1.23	40.0
1.07	80.0
1.05	Leachate

TABLE F-5.10 VOID RATIO - PRESSURE RESULTS

(CELL C-5.10)

VOID RATIO (dimensionless)	PRESSURE (kPa)
8.36	1.0
7.22	10.0
5.46	20.0
4.47	40.0
3.63	80.0
3.30	Leachate

TABLE F-5.11 VOID RATIO - PRESSURE RESULTS

(CELL C-5.11)

VOID RATIO (dimensionless)	PRESSURE (kPa)
8.42	1.0
7.45	10.0
5.40	20.0
4.56	40.0
3.69	80.0
3.03	Leachate

TABLE F-6.1 VOID RATIO - PRESSURE RESULTS
(CELL C-6.1)

VOID RATIO (dimensionless)	PRESSURE (kPa)
16.43	1.0
12.22	10.0
10.28	20.0
8.39	40.0
6.21	Leachate
4.26	80.0 + Leachate
4.19	de-ionized water

TABLE F-6.2 VOID RATIO - PRESSURE RESULTS
(CELL C-6.2)

VOID RATIO (dimensionless)	PRESSURE (kPa)
13.06	1.0
10.06	10.0
8.24	20.0
6.68	40.0
5.18	Leachate
3.67	80.0 + Leachate
3.63	de-ionized water

TABLE F-6.3 VOID RATIO - PRESSURE RESULTS

(CELL C-6.3)

VOID RATIO (dimensionless)	PRESSURE (kPa)
9.42	1.0
8.45	10.0
6.18	20.0
5.16	40.0
4.43	Leachate 1
3.28	80.0
3.25	Leachate 2

TABLE F-6.4 VOID RATIO - PRESSURE RESULTS

(CELL C-6.4)

VOID RATIO (dimensionless)	PRESSURE (kPa)
7.50	1.0
5.44	10.0
4.59	20.0
3.92	40.0
3.59	Leachate

TABLE F-6.5 VOID RATIO - PRESSURE RESULTS

(CELL C-6.7)

VOID RATIO (dimensionless)	PRESSURE (kPa)
15.01	1.0
11.15	10.0
9.88	20.0
8.07	40.0
6.17	80.0
4.68	160.0
3.26	Leachate
2.79	240.0 + Leachate

TABLE F-6.6 VOID RATIO - PRESSURE RESULTS

(CELL C-6.8)

VOID RATIO (dimensionless)	PRESSURE (kPa)
12.71	1.0
9.54	10.0
8.47	20.0
6.87	40.0
5.33	80.0
3.97	160.0
2.94	Leachate
2.51	240.0 + Leachate

TABLE F-6.7 VOID RATIO - PRESSURE RESULTS

(CELL C-6.9)

VOID RATIO (dimensionless)	PRESSURE (kPa)
9.90	1.0
7.45	10.0
6.70	20.0
5.43	40.0
4.24	80.0
3.25	160.0
2.67	Leachate
2.33	240.0 + Leachate

TABLE F-6.8 VOID RATIO - PRESSURE RESULTS

(CELL C-6.10)

VOID RATIO (dimensionless)	PRESSURE (kPa)
7.10	1.0
4.84	10.0
4.36	20.0
3.74	40.0
3.21	80.0
2.61	160.0
2.38	Leachate
2.16	240.0 + Leachate

TABLE F-6.9 VOID RATIO - PRESSURE RESULTS

(CELL C-6.11)

VOID RATIO (dimensionless)	PRESSURE (kPa)
5.12	1.0
3.72	10.0
3.43	20.0
3.07	40.0
2.72	80.0
2.31	160.0
2.25	Leachate
2.09	240.0 + Leachate

TABLE F-6.10 VOID RATIO - PRESSURE RESULTS

(CELL C-6.12)

VOID RATIO (dimensionless)	PRESSURE (kPa)
3.82	1.0
3.01	10.0
2.86	20.0
2.63	40.0
2.40	80.0
2.12	160.0
2.10	Leachate
2.00	240.0 + Leachate

TABLE F-6.11 VOID RATIO - PRESSURE RESULTS

(CELL C-6.5)

VOID RATIO (dimensionless)	PRESSURE (kPa)
5.33	1.0
3.82	10.0
3.53	20.0
3.12	40.0
3.03	Leachate

TABLE F-6.12 VOID RATIO - PRESSURE RESULTS

(CELL C-6.6)

VOID RATIO (dimensionless)	PRESSURE (kPa)
3.79	1.0
2.97	10.0
2.84	20.0
2.60	40.0
2.57	Leachate

TABLE F-7.1 VOID RATIO - PRESSURE RESULTS

(CELL C-7.1)

VOID RATIO (dimensionless)	PRESSURE (kPa)
2.17	1.0
1.84	10.0
1.67	20.0
1.45	40.0
1.21	80.0
1.20	Leachate
1.02	160.0 + Leachate

TABLE F-7.2 VOID RATIO - PRESSURE RESULTS

(CELL C-7.2)

VOID RATIO (dimensionless)	PRESSURE (kPa)
2.04	1.0
1.70	10.0
1.54	20.0
1.36	40.0
1.16	80.0
1.15	Leachate
0.98	160.0 + Leachate

TABLE F-7.3 VOID RATIO - PRESSURE RESULTS

(CELL C-7.3)

VOID RATIO (dimensionless)	PRESSURE (kPa)
1.64	1.0
1.51	10.0
1.36	20.0
1.21	40.0
1.00	80.0
0.99	Leachate
0.84	160.0 + Leachate

TABLE F-7.4 VOID RATIO - PRESSURE RESULTS

(CELL C-7.4)

VOID RATIO (dimensionless)	PRESSURE (kPa)
2.79	1.0
2.02	10.0
1.91	20.0
1.63	40.0
1.36	80.0
1.34	Leachate
1.14	160.0 + Leachate

TABLE F-7.5 VOID RATIO - PRESSURE RESULTS

(CELL C-7.5)

VOID RATIO (dimensionless)	PRESSURE (kPa)
3.24	1.0
2.34	10.0
2.15	20.0
1.82	40.0
1.54	80.0
1.51	Leachate
1.30	160.0 + Leachate

TABLE F-7.6 VOID RATIO - PRESSURE RESULTS

(CELL C-7.6)

VOID RATIO (dimensionless)	PRESSURE (kPa)
2.59	1.0
1.90	10.0
1.77	20.0
1.50	40.0
1.25	80.0
1.21	Leachate
1.03	160.0 + Leachate

TABLE F-8.1 VOID RATIO - PRESSURE RESULTS

(CELL C-8.1)

VOID RATIO (dimensionless)	PRESSURE (kPa)
4.96	1.0
3.86	10.0
3.22	20.0
2.73	40.0
2.25	80.0
2.14	Leachate
1.77	160.0 + Leachate
1.42	320.0 + Leachate
1.24	560.0 + Leachate

TABLE F-8.2 VOID RATIO - PRESSURE RESULTS

(CELL C-8.2)

VOID RATIO (dimensionless)	PRESSURE (kPa)
3.37	1.0
2.76	10.0
2.49	20.0
2.17	40.0
1.85	80.0
1.84	Leachate
1.61	160.0 + Leachate
1.37	320.0 + Leachate
1.22	560.0 + Leachate

TABLE F-8.3 VOID RATIO - PRESSURE RESULTS

(CELL C-8.3)

VOID RATIO (dimensionless)	PRESSURE (kPa)
5.55	1.0
4.06	10.0
3.57	20.0
3.07	40.0
2.51	80.0
2.32	Leachate
1.86	160.0 + Leachate
1.51	320.0 + Leachate
1.29	560.0 + Leachate

TABLE F-8.4 VOID RATIO - PRESSURE RESULTS

(CELL C-8.4)

VOID RATIO (dimensionless)	PRESSURE (kPa)
11.54	1.0
8.15	10.0
7.29	20.0
5.99	40.0
4.78	80.0
3.40	Leachate (3 types)
2.56	160.0 + Leachate
1.91	320.0 + Leachate

APPENDIX G - Sedimentation Results

TABLE G-1.1

FSDP VERSUS Ca AND Na CONCENTRATIONS

(Soil: kaolin to bentonite = 4 to 1, HAc = 250 mg/l)

FSDP	Calcium (moles/l)	Sodium (moles/l)
37.3	0.009	0.017
45.1	0.036	0.017
46.7	0.063	0.017
47.2	0.090	0.017
40.0	0.009	0.068
46.7	0.036	0.068
48.8	0.063	0.068
49.3	0.090	0.068
41.3	0.009	0.119
46.9	0.036	0.119
48.8	0.063	0.119
49.3	0.090	0.119
41.3	0.009	0.170
49.1	0.036	0.170
49.3	0.063	0.170
49.3	0.090	0.170

TABLE G-1.2

FSDP VERSUS Ca AND Na CONCENTRATIONS

(Soil: kaolin to bentonite = 13 to 7, HAc = 250 mg/l)

FSDP	Calcium (moles/l)	Sodium (moles/l)
42.1	0.009	0.017
57.1	0.036	0.017
61.4	0.063	0.017
62.2	0.090	0.017
45.9	0.009	0.068
58.9	0.036	0.068
62.2	0.063	0.068
63.2	0.090	0.068
49.5	0.009	0.119
59.3	0.036	0.119
63.6	0.063	0.119
63.4	0.090	0.119
52.3	0.009	0.170
60.4	0.036	0.170
62.2	0.063	0.170
62.9	0.090	0.170

TABLE G-1.3

FSDP VERSUS Ca AND Na CONCENTRATIONS

(Soil: kaolin to bentonite = 1 to 1, HAc = 250 mg/l)

FSDP	Calcium (moles/l)	Sodium (moles/l)
43.4	0.009	0.017
63.7	0.036	0.017
68.4	0.063	0.017
69.7	0.090	0.017
50.0	0.009	0.068
65.8	0.036	0.068
69.7	0.063	0.068
70.0	0.090	0.068
53.7	0.009	0.119
65.8	0.036	0.119
68.7	0.063	0.119
69.9	0.090	0.119
56.3	0.009	0.170
67.0	0.036	0.170
69.1	0.063	0.170
69.7	0.090	0.170

TABLE G-1.4

FSDP VERSUS Ca AND Na CONCENTRATIONS

(Soil: kaolin to bentonite = 4 to 1, HAc = 250 mg/l)

FSDP	Calcium (moles/l)	Sodium (moles/l)
43.8	0.009	0.017
65.1	0.036	0.017
69.7	0.063	0.017
72.2	0.090	0.017
47.9	0.009	0.068
66.5	0.036	0.068
71.4	0.063	0.068
71.9	0.090	0.068
52.8	0.009	0.119
66.8	0.036	0.119
70.7	0.063	0.119
71.8	0.090	0.119
55.3	0.009	0.170
68.5	0.036	0.170
69.8	0.063	0.170
71.8	0.090	0.170

TABLE G-2.1

FSDP VALUES OBTAINED IN A CENTRAL COMPOSITE DESIGN INVOLVING 4 VARIABLES (a)

% Bentonite	Na Conc. (mg/l)	Ca Conc. (mg/l)	HAc Conc. (mg/l)	FSDP
36	5,500	3,163	5,500	59.1
36	1,000	1,000	5,500	49.6
36	1,000	10,000	5,500	57.1
36	10,000	10,000	5,500	62.0
36	10,000	1,000	5,500	53.9
65	5,500	3,163	10,000	67.5
65	5,500	3,163	1,000	66.8
20	5,500	3,163	10,000	47.7
20	5,500	3,163	1,000	47.2

TABLE G-2.2

FSDP VALUES OBTAINED IN A CENTRAL COMPOSITE DESIGN INVOLVING 4 VARIABLES (b)

% Bentonite	Na Conc. (mg/l)	Ca Conc. (mg/l)	HAc Conc. (mg/l)	FSDP
36	10,000	3,163	10,000	60.5
36	1,000	3,163	1,000	57.1
36	10,000	3,163	1,000	59.1
36	1,000	3,163	10,000	57.1
36	5,500	3,163	5,500	58.75
65	5,500	10,000	5,500	71.9
20	5,500	1,000	5,500	41.3
65	5,500	1,000	5,500	59.9
20	5,500	10,000	5,500	49.6

TABLE G-3.1

"Free Swell" (V_d) Values at Different Temperatures

SOIL	LEACHATE	"V_d" at 0 C (ml)	"V_d" at 25 C (ml)
bentonite = 40% Kaolinite = 60%	Ca = 0.004 M plus base	41.4	40.8
		41.0	41.2
		41.0	41.2
	Ca = 0.045 M plus base	22.4	22.2
		22.4	22.2
		22.5	22.6

TABLE G-3.2

"Free Swell" (V_d) Values at Different Temperatures

SOIL	LEACHATE	"V_d" in 100 ml Cylinder (ml)	"V_d" in 250 ml Cylinder (ml)
bentonite = 40% Kaolinite = 60%	distilled water	91.2	91.0
		91.2	91.0
	Ca = 0.045 M plus base	28.4	28.2
		28.2	28.0

APPENDIX H - Photomicrographs

Fig. H-1.1
Photomicrograph of the 60% kaolin, 40% bentonite soil sample kept in deionized water, viewed normal to the direction of consolidation. (mag. = 200x)

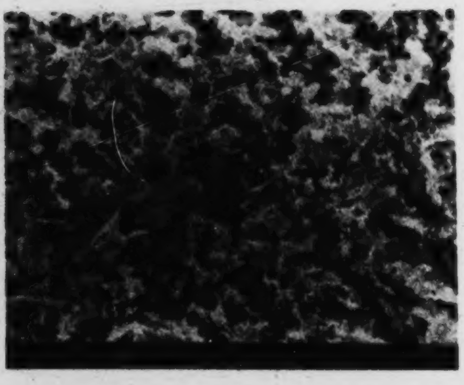

Fig. H-1.2
A higher magnification of the same soil sample showing shrinkage induced cracks. (mag. = 900x)

Fig. H-1.3
Photomicrograph of the same soil showing oriented kaolin plates. (mag. = 9000x)

Fig. H-1.4
85% bentonite, 15% Ca mont. sample, viewed normal to the direction of consolidation. (mag. = 9000x)

Fig. H-1.5
Photomicrograph of the 60% kaolin, 40% bentonite soil sample permeated with 10,000, ppm $CaCl_2$. Viewed normal to consolidation. (mag. = 100x)

Fig. H-1.6
A higher magnification of the same soil sample.

(mag. = 900x)

Fig. H-1.7
Photomicrograph of the same soil showing loosely flocculated arrangement.
(mag. = 9000x)

Fig. H-1.8
Photomicrograph of the same soil, prepared using Freon and viewed parallel to the direction of consolidation.
(mag. = 9000x)

Fig. H-1.9
A higher magnification of the sample in fig. H-1.8. (mag. = 2000x)

Fig. H-1.10
Photomicrograph of a soil sample containing 25% bentonite, 75% Ca montmorillonite treated with standard leachate (mag. = 100x).

Fig. H-1.11
Photomicrograph of the same sample at a higher magnification.. (mag. = 1000x)

Fig. H-1.12
Photomicrograph of the same soil, showing the dense soil structure. (mag. = 9000x)

Fig. H-1.13
Photomicrograph from the 60% kaolin, 40% bentonite soil sample permeated with 5,000 ppm $CaCl_2$, prepared using Freon, viewed normal to consolidation direction. (mag. = 2000x)

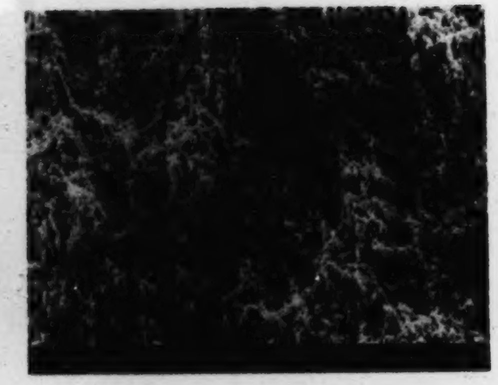

Fig. H-1.14
Photomicrograph of the same soil sample, at a lower magnification. (mag. = 200x)

Fig. H-1.15
Photomicrograph of the 60% kaolin and 40% bentonite soil, permeated with 5,000 ppm $CaCl_2$, viewed normal to direction of consolidation (mag. = 100x).

Fig. H-1.16
Photomicrograph of the same soil sample at a higher magnification. (mag. = 1000x)

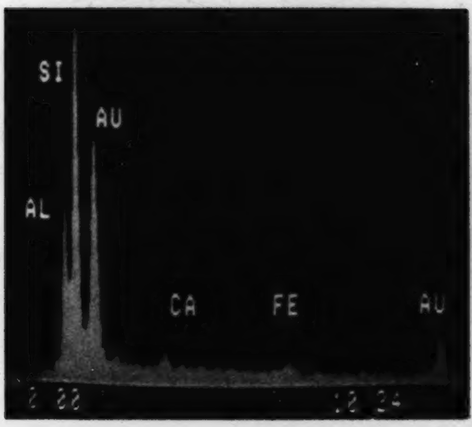

Fig. H-2.1
Elemental analysis of 60% kaolin, 40% bentonite soil sample permeated with 5,000 ppm $CaCl_2$.

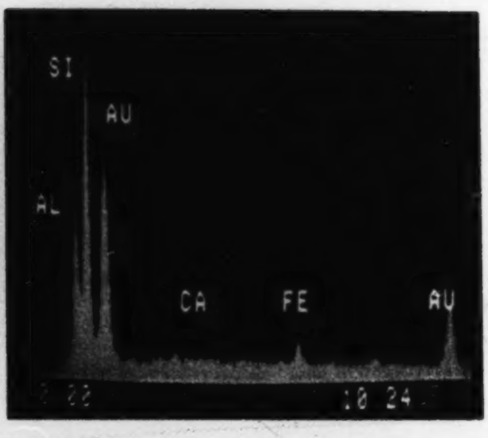

Fig. H-2.2
Elemental analysis of the same soil treated with deionized water.

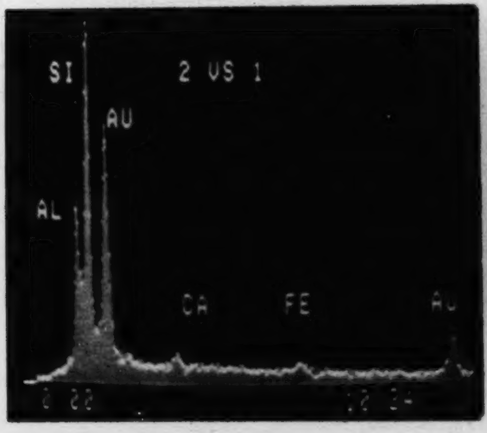

Fig. H-2.3
Elemental analysis of soil in fig. H-2.1 and 60% kaolin and 40% bentonite, treated with 10,000ppm $CaCl_2$.

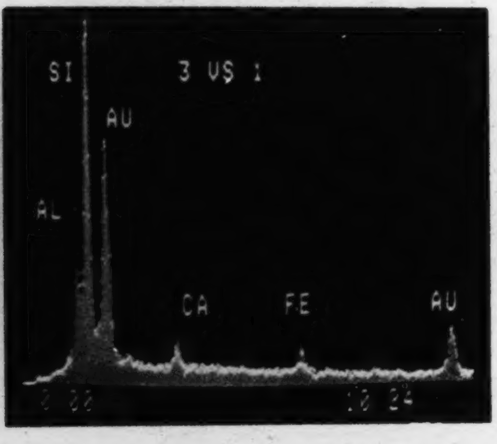

Fig. H-2.4
Elemental analysis of soil in fig. H-2.1 and 30% kaolin and 70% bentonite, treated with standard leachate.